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WT-116 (Ref.)

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OPERATION GREENHOUSE

SCIENTIFIC DIRECTOR'S REPORT

ANNEX 3.1 U. S. ARMY STRUCTURES

APPENDIX 1

NUCLEAR EXPLOSIONS

1951

[redacted]

[redacted]

Defense Nuclear Agency
6801 Telegraph Road
Alexandria, Virginia 22310-3398

SSTS

28 June 1995

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FOR THE DIRECTOR

JOSEPHINE B. WOOD
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Scientific Director's Report of Atomic Weapon Tests at Eniwetok, 1951

Annex 3.1

U. S. Army Structures

Appendix 1

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U. S. ARMY STRUCTURES

Appendix 1
Design of Test Structures

by

AMMANN & WHITNEY, CONSULTING ENGINEERS
(Under Contract W49-129-Eng 148 with
the Chief of Engineers, Department of
the Army)

Approved by
SHERWOOD B. SMITH
Director, Program 3

Ammann & Whitney, Consulting Engineers
New York, N. Y.
September 1951

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[redacted]

PREFACE

This Appendix is essentially the final report submitted to the Chief of Engineers on 15 June, 1950 by the firm of Ammann and Whitney, Consulting Engineers, 76 Ninth Avenue, New York 11, N. Y., in partial fulfillment of Contract W49-129-Eng-148. Their final report contained the following items as specified in paragraph C-6 of their contract:

a. A description and discussion of design procedures and methods for use in the design of blast resistant structures, including the source of material, the basic assumptions, and the expected accuracy of the assumptions.

b. A description of each of the buildings in the test structure including complete plans and specifications.

c. A complete presentation of the pressure-time curves used as the design loads on the various buildings.

d. Summary sheets tabulating the pressures, materials, dimensions, and the unit stresses for the various panels, walls, roofs and frames.

e. Samples of the computations for the test building illustrating various detailed procedures.

f. A summary of the proposed instrumentation with emphasis on the type of information desired, with drawings and charts showing the location and required sensitivity of the instruments.

g. An outline of a recommended general and detailed procedure for the analysis of the field measurements and the modification of the design methods.

[redacted]

h. An outline of a tentative procedure for designing blast resistant structures including preliminary recommendations on preferable construction materials and types of framing.

i. A bibliography of technical literature.

Distribution of printed and bound copies of the Ammann & Whitney report was made by the Office, Chief of Engineers to certain interested agencies and individuals subsequent to its publication in June, 1950. It has been republished as a part of the Final Greenhouse Report because numerous references are made to it in various portions of the Army Structures Test Report which require that it be available to the many recipients of the Greenhouse Reports who were not included in the original distribution of the Ammann & Whitney report by the Office, Chief of Engineers. Further, it is considered appropriate that the criteria, design assumptions and other data contained therein be made a matter of record in connection with the Greenhouse tests since these data are not contained elsewhere in the Army Structures Test Report.

Due to its earlier publication as a separate report, this Appendix does not follow the format established for the Greenhouse reports. In order that any references may be applicable to those copies of the report distributed by the Chief of Engineers as well as to this Appendix, the report has been reproduced herein exactly as initially published, with the exception of certain corrections which have been made. Also, drawings showing later revision dates have been used in lieu of those contained in the report as published initially.

It should be noted that the analysis and discussion relative to the steel mill building design on page 18 and in Part A 2.3.6 of Appendix 2 of the Ammann and Whitney Report have not been deleted although the steel mill buildings were eliminated from the test program after the design had been made.

15 September, 1951

[redacted]

CONTENTS*

*For detailed contents see Appendix 9.

[redacted]

CONTENTS (Continued)

CONTENTS (Contents)

[redacted]

DESIGN OF STRUCTURES EXPOSED TO HEAVY BLAST LOADS

PART I. INTRODUCTION

Contents

1.1 Conditions and Requirements of Program for the Corps of Engineers, Protective Construction Branch

1.1.1 Scope of Service

The service rendered by the contractor included providing detailed plans and specifications for the varied structures as described by the test program of the Corps of Engineers, Protective Construction Branch. Additional details of the service included the checking of shop drawings, preparation of an estimate of construction costs, and aid in the preparation of drawings showing the location and mounting of the recording instruments.

From the time the requirements of the test program were first presented, the contractor has worked in close collaboration with the government consultants and under the general direction of the Protective Construction Branch of the Corps of Engineers. The contractor also received, at various times, test data, reports, and constructive suggestions from the same groups. The reports of the investigations conducted by the Massachusetts Institute of Technology for the first part of the Corps of Engineers test program were particularly useful in this respect.

The close cooperation of Dr. Curtis W. Lampson in providing the necessary loading curves also played an essential part in the successful progress of the program.

A number of conferences were held during the execution of the contract, at which time the Corps of Engineers and the consultants reviewed the methods and procedures being used in the design, offering their suggestions, criticisms, and general approval.

1.1.2 Development of the Contract

The original program formulated by the Corps of Engineers consisted of a number of structures including a multi-story building, three stories in height, 50 feet wide and 190 feet in length; a one-story steel mill building 50 feet x 200 feet; and a two-story wood barracks building, approximately 30 feet x 80 feet. These buildings were to consist of the structural framing and the closure walls, without architectural trim or finishes and without utilities.

The contract was later modified by eliminating the steel and wood buildings and by adding a varied type personnel shelter. The multi-story building remained as originally intended except for minor changes which permitted fuller utilization of its various sections for test purposes. This to an extent compensated for the elimination of the steel mill building and other independent test units.

1.1.3 Purpose of the Investigation

The primary purpose of this investigation is to develop procedures for the structural design of buildings to resist the heavy impulsive loads of A-Bomb blasts.

Such procedures are needed in order to determine the types, proportions, and costs of buildings suitable for resisting such loads and to indicate the practical limits of blast resistance. These methods will also provide a rational approach to the problem of estimating bomb damage of an assumed bomb on any specific structure.

In order to carry out the contract, it has been necessary to develop new methods for analyzing structures under impulsive loading. These procedures include the effect of the inertia of the mass of the structure and the energy absorbing ability of the materials in the plastic range. Prior to the development of the basic assumptions, a study was made of available reports on tests of dynamic properties of structural materials and members. The ultimate strength of reinforced concrete members has been estimated by use of the plastic theory which is much more realistic than the standard "straight line theory" currently used in the design of ordinary buildings.

A study was made of the effect of A-bomb blasts on buildings in Japan and large bombs in Europe for the purpose of reconciling the proposed design procedure with past experience.

Instead of designing the structure in a conventional manner so that the stresses in all parts are well below the elastic limit or yield stress, it has been assumed, in order to take advantage of the absorption of blast energy by plastic action of the building frame, that certain structural members will be stressed beyond their elastic limit. To obtain this condition, it is intended to permit the maximum distortion of the frames and walls which is possible without danger of collapse or irreparable damage to the usefulness of the building.

In order to check and refine the design procedures developed, they were applied concurrently to the design of a group of buildings which are to be tested by exposure to an A-bomb blast. Instrumentation of the test structures for the purpose of deriving useful information on the basic assumptions is recommended in this report and methods of analyzing of the test data are considered.

Some general conclusions have been drawn as to proper procedure in designing buildings for the purpose under consideration, but general and comprehensive rules must await a thorough post-test analysis of the behavior of the structures and, from this, an examination of the suitability of the adopted design criteria.

1.2 Types of Test Structures

Since the most important modern construction materials are structural steel and reinforced concrete, buildings with frames of each are included in the test structures. No masonry bearing walls were used because of their very poor blast resistance though some reinforced brick panels are included to obtain general information. Floor slabs are in all cases monolithic reinforced concrete because of its continuity and superior toughness. A great variety of materials and designs are used for the wall panels in an attempt to secure information which will be helpful in designing any type of curtain wall which may later appear to be practical.

One of the buildings has been designed as a shear-wall structure of reinforced concrete with the solid cross walls at each end to resist the horizontal thrust of the blast. This type may prove very economical under favorable conditions, but because of its greater rigidity its action under dynamic loading is quite different from that of the flexible frames. It was therefore felt important to make a full scale test of its action and a building of this type was included as one of the requirements furnished to the contractor.

Tests are also to be made on a semi-buried personnel shelter structure utilizing various types of steel and concrete units.

1.2.1 Frame Buildings

(a) Structural Steel and Reinforced Concrete Frames. Both steel and concrete frame buildings were included in accordance with the original scope of the program. Though it developed that the less massive steel buildings would probably cost considerably more than the equivalent strength reinforced concrete buildings, these buildings provide several useful functions which warrant their construction.

The walls of the steel buildings are of steel construction, much lighter than concrete walls, and are made discontinuous at each floor level to simplify future interpretation of the tests and to facilitate evaluation of the design procedures. The entire lateral blast impulse of these buildings is therefore carried solely by the frame action of the columns and girders. The concrete buildings, on the contrary, have reinforced concrete walls continuous from top to bottom which give considerable assistance to the frame by adding to both the mass and the resistance to lateral motion.

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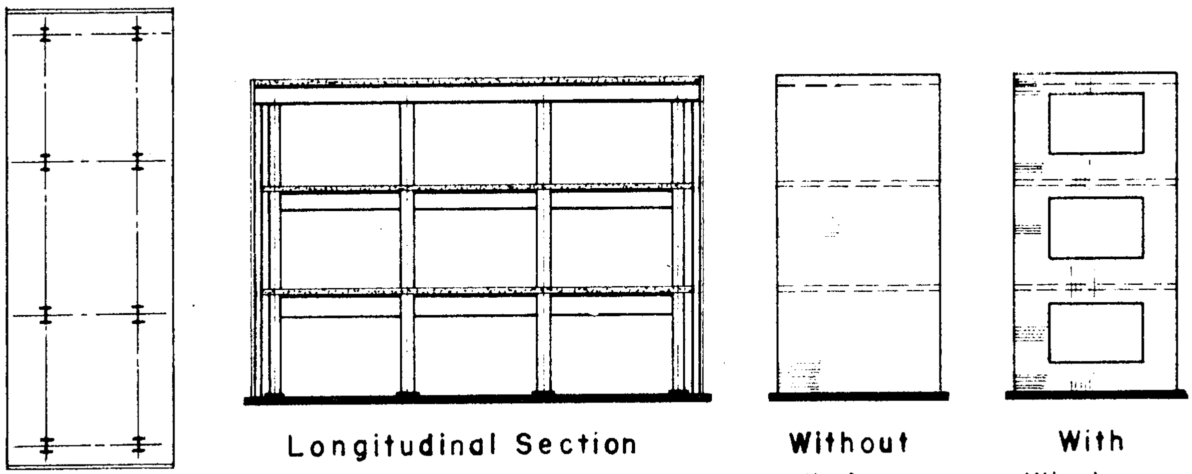
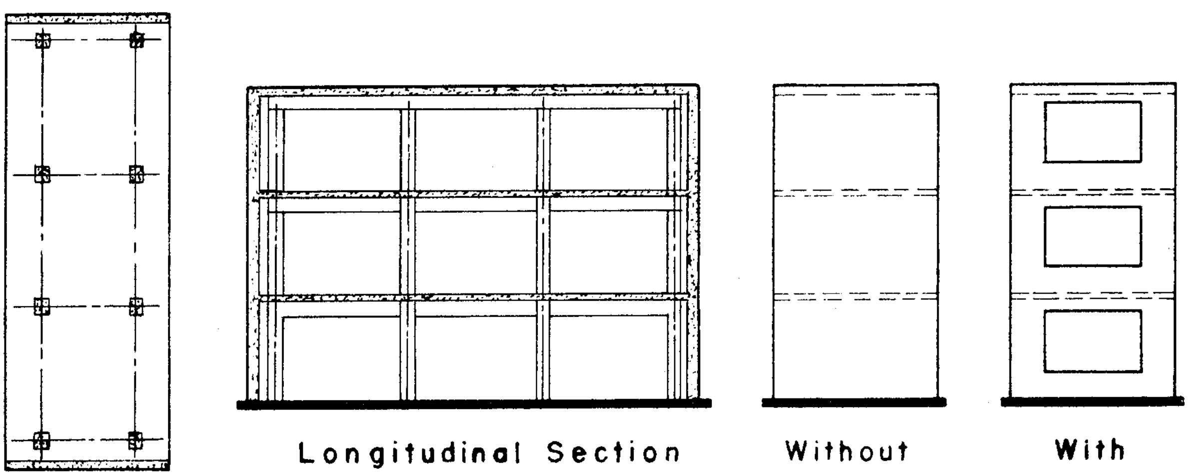


STEEL FRAME BUILDING



CONCRETE FRAME BUILDING
FIG. I.2.I-I

The steel frame building is expected to provide test data on the behavior of structural steel in the plastic range, to test the adequacy of high yield-point ductile welded connections under blast loads, and to indicate the comparative cost of such construction. Tests of the concrete buildings, heavier, stiffer and more economical than the steel buildings, will yield very valuable information regarding the action of reinforced concrete under suddenly applied thrust and bending, and on many other aspects of its behavior, such as its bond and shear strength, which are not well understood at present even under static loading.

(b) Buildings With and Without Windows. In the A-bombings of Japan, conventional multi-storied buildings of both steel and reinforced concrete fared rather well when they were 2500 feet or more from "ground zero." In such structures, the windows and light curtain walls evidently failed without substantial resistance, reducing the horizontal impulsive load by reduction of the effective resisting area before large deformations had occurred. While these openings are helpful in reducing damage to the frame and reducing danger of overturning, such construction offers little protection to personnel and contents of the building. In cases where windows are not actually required by the occupancy of the building, the question therefore arises as to whether or not windowless blast-proof wall panels can be provided without unreasonable expense or damage to the frame.

For this reason independent sections of both the steel and concrete buildings were designed with and without windows to test the difference in behavior between these two arrangements offering widely different degrees of protection to the structure, its contents, and its personnel.

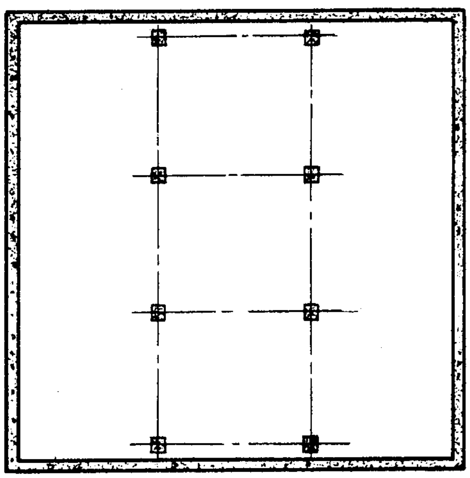
Frames of equal strength are used for the buildings with and without window openings, so that they will provide horizontal blast load capacity through a wider range of loading intensity, will record the action under different degrees of plastic deformation, and will provide an almost direct comparison of the impulse applied to each type. The increase in range is particularly important because of probable inaccuracies in estimating the intensity of blast pressure for which the test structures are designed.

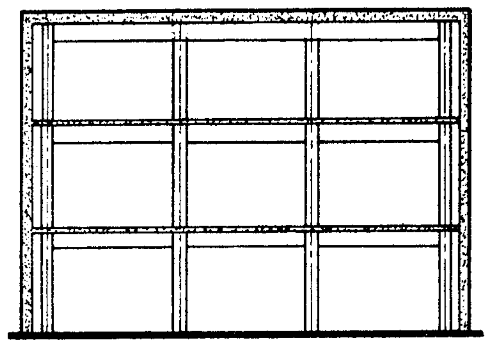
1.2.2 Shear Wall Building

As any reinforced concrete building having blast resistant walls would have such walls on all sides, a new element resisting lateral motion is introduced into the structure. The stiff floors will transmit horizontal load to the relatively rigid side walls before sufficient deformation has occurred to load the more flexible frames. This produces a radically different distribution of stresses in all parts of the structure and its foundations from that assumed in the frame buildings. If it is possible to provide such rigid cross walls at suitable intervals, they may provide the required lateral strength at lower cost than frames. They are however subjected to much higher shears under dynamic load because of their lack of flexibility.

[redacted]

In order to develop design procedures and test data for this type of structure, the central building of the seven buildings in the multi-story group was designed as a shear wall structure with four solid walls on an elastic foundation.





Longitudinal Section

Sectional Plan

SHEAR WALL BUILDING

FIG.1.2.2-1

1.2.3 End Cell Buildings

The end buildings in the multi-story group serve to isolate the interior buildings from the end blast effects. These buildings, consisting of rigid reinforced concrete cells, also provide an opportunity for the testing of various types of panels built into the exposed faces. These panels include one-way and two-way reinforced concrete slabs, light gage steel siding, corrugated asbestos siding, and plain and reinforced brick panels. A wide range of panel strengths is used in order to insure, insofar as possible, that regardless of the range in blast intensity, some panels will carry the load within the elastic range while others will exhibit plastic

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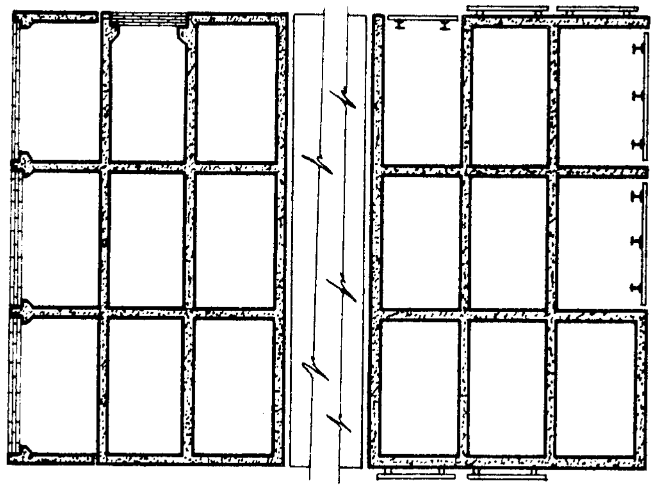
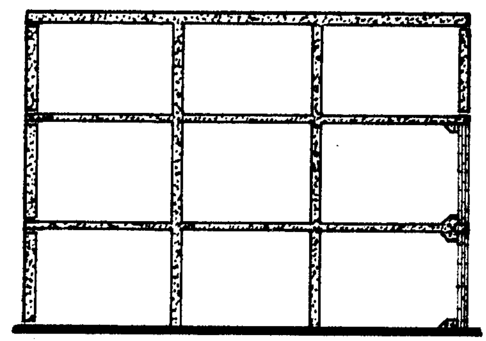
yield. It is hoped that the variety of test panels is sufficient to permit the appraisal of any practical type of panel construction after the test results are analyzed.



Sectional Plan



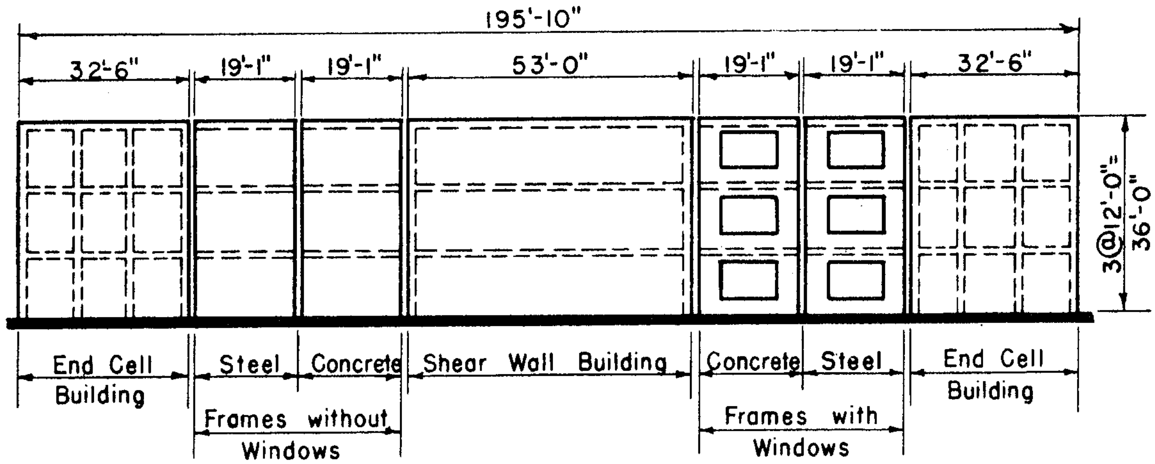
Longitudinal Section
(Interior Typical)

END CELL BUILDINGS
FIG. 1.2.3-1

1.2.4 Arrangement of Multi-Story Buildings

For economy, the various test buildings are arranged to minimize the number of structures needed to seal the blast from the interior of the different buildings. To accomplish this, concrete and steel frame buildings with windows are placed side by side between the end cell building and the central shear wall building. The two frame buildings without windows are symmetrically and similarly placed on the other side of the shear wall building. The arrangement of the different types of multi-story buildings and their general dimensions are in figure 1.2.4-1.

[redacted]

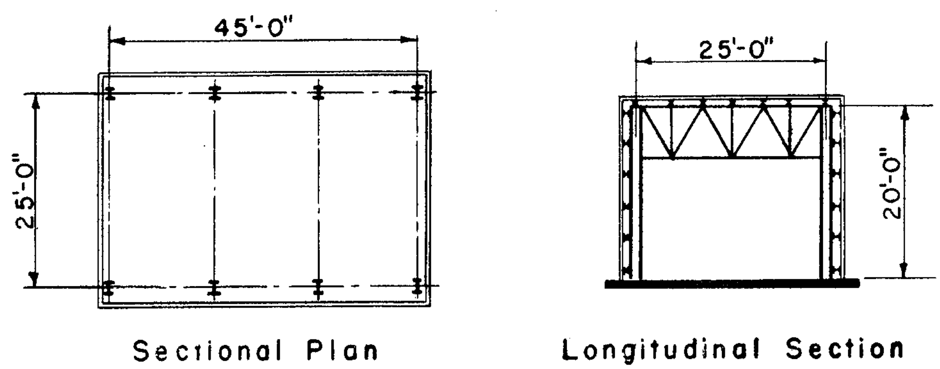


ARRANGEMENT OF MULTI-STORY BUILDINGS

FIG. 1.2.4-1

1.2.5 Steel Mill Buildings

Careful consideration was given to the testing of a conventional type one-story steel mill building with a comparatively long roof span. Detailed designs were prepared which will be included and described in this report. It was found to be very difficult to provide blast resistance for this type because of its large area of exposure and lack of mass to absorb the dynamic blow. Because of its extremely high cost and the limitation of funds, the steel mill buildings will not be built for test. Figure 1.2.5-1 shows the general dimensions of the steel mill building assumed for analysis and design.



STEEL MILL BUILDING

FIG. 1.2.5-1

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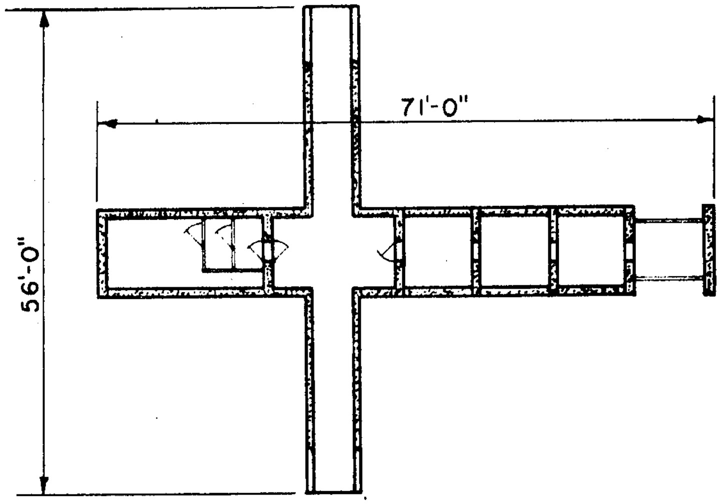
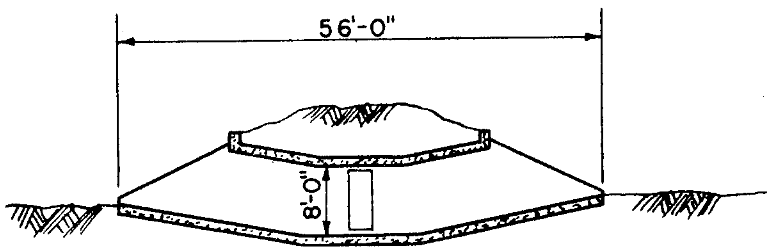
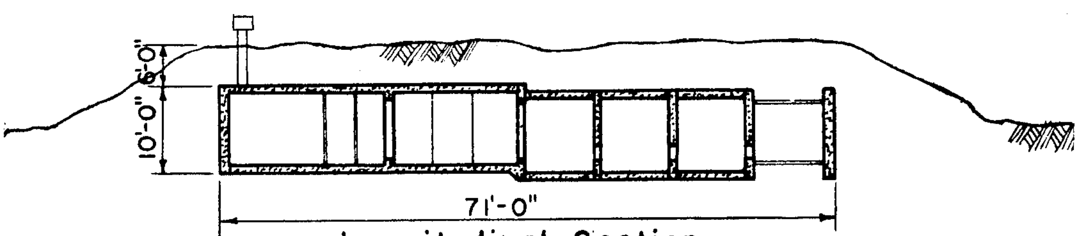


Sectional Plan



Transverse Section



Longitudinal Section

PERSONNEL SHELTER

FIG. 1.2.6-1

[redacted]

1.2.6 Personnel Shelter

Included in the test is a personnel shelter buried in the ground with sufficient earth cover to provide protection against radiation. It is divided in two parts by a tunnel entranceway open at both ends to avoid trapping the blast. One part is of cast-in-place reinforced concrete of rectangular cross section designed to resist the blast loading without substantial deformation. The other part is composed of short sections of precast concrete of varying strengths, and a corrugated section of steel circular pipe. The general arrangement is shown in figure 1.2.6-1.

1.3 Range of Strength of Test Structures

Formulas used in structural design are inexact even for the simplest static loading conditions because of unpredictable variations in both the characteristics of the materials involved and inaccuracies in design theories. For example the tensile strength of steel purchased under a standard specification may vary twenty percent or more. These physical properties are further affected by the speed of straining although an insufficient amount of experimental work has been completed on this phase to permit accurate prediction of the behavior of materials and structural members under an impulsive loading such as a bomb blast.

Analytical procedures have been developed by this contractor for calculating the deformation of structural frames and members subjected to blast loading. It is believed that these procedures, which are based on a study of available data, will prove to be satisfactory, but their application to any specific case depends on many assumptions and design simplifications which will necessarily introduce uncertainties, at least until they are tested and modified by further laboratory research and tests on prototypes.

Further uncertainty is introduced by the fact that the assumed intensity of blast loading involves an estimate of the bomb's "explosive equivalent," and the application of shock wave theory and the elaborate and, as yet, somewhat uncertain diffraction theory. As a result there are expected to be certain errors in the sets of time-pressure curves which have been furnished to the contractor and which have necessarily been used by him in estimating the loads.

Taking these factors into consideration, an attempt has been made, in designing the test structures, to include a range of strengths which will be wide enough to assure that the A-bomb blast will produce action providing the desired information despite the uncertainties. Care has been exercised to avoid insofar as possible either complete collapse of the structure, which might destroy all evidence except measurements recorded prior to failure, or a too conservative design in which the deflections would all lie within the elastic range, thereby throwing little light on the behavior in the plastic range, and on the probable modes of failure.

[redacted]

After thorough consideration, the following design criteria were selected as being the ones which would probably give the most valuable test results.

(a) The framed buildings are designed for a maximum floor to floor displacement as follows:

Building	Percent of Theoretical Pressure		
	65%	100%	135%
Concrete framed buildings - without windows	2"	5"	14"
Concrete framed buildings - with windows	1"	2½"	6"
Steel framed buildings - without windows	2½"	4½"	9"
Steel framed buildings - with windows	1"	2"	3"

(b) The roof slabs were originally designed for a deflection of L/32 under 125% of the theoretical blast load. Due to the reduction in the estimated blast pressures and a shift in the building locations, the deflections of these members are now expected to be near the elastic range.

(c) The front wall panels in general are designed for a deflection of L/32 at a pressure of 135% of the estimated blast pressures. The rear walls are expected to carry the loads at or near the elastic range.

(d) The various test panels on the end buildings were originally designed for deflections of 1/32 of their span under a range of loading varying from 50% to 150% of the original estimated blast pressure. For the same reason as mentioned in (b) the range is now increased to cover 60% to 175% and in some cases higher percentages of the blast load.

In covering a range from 50% to 150% of the estimated blast pressures in the design of the panels covering the faces of the end buildings, the probability is accepted that some will either fail completely or will work only in the elastic range. There seems to be no alternative which would insure that at least part of the structural elements will undergo the desired kind of action (plastic yield without failure).

(e) The poured-in-place portion of the shelter is designed to carry the blast pressure with small total deflections and anticipates a minimum evidence of plastic yield on the interior exposed surfaces. The precast and corrugated iron sections are designed in varied strengths in an attempt to obtain design information on the load distribution to rigid and flexible members buried in the ground.

A more complete listing of the expected deflections is shown in Appendix 3.

[redacted]

1.4 Design Procedures

The analysis of the major part of the structural members under dynamic loading has been effected by step-by-step procedures, although semi-graphical methods were found convenient in some cases. These procedures have proved to be simple and effective when applied to the design of the test buildings, the most lengthy solution being that for the frame building with the front and rear walls participating in resisting lateral bending. Possible simplifications are also indicated for this case but were not carried to completion as it is believed that a better evaluation will be possible after the basic assumptions have been confirmed by the tests.

The proposed tests will permit the observation of the effect of actual pressures on structures built of materials of known physical characteristics. The design theory can then be checked and refined and reviewed for possible simplification. With that accomplished, the theory may be applied to structures of widely different proportions to determine the bomb resistance of existing buildings or to design new buildings for any specific set of conditions.

1.5 Direction

This contract has been executed under the direction of the Corps of Engineers and their consultants who have reviewed the major principles involved in the work and offered very helpful suggestions.

The following government employed consultants collaborated with the contractor.

Dr. Nathan M. Newmark,	Research Professor of Structural Engineering, University of Illinois.
Dr. John D. Wilbur,	Head, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology.
Dr. Charles H. Norris,	Professor, Massachusetts Institute of Technology.
Dr. Robert J. Hansen,	Associate Professor, Massachusetts Institute of Technology.

The data concerning the blast pressures was furnished by Dr. Curtis W. Lampson, Chief Engineering Division, Ballistic Research, Ordinance Department, U.S.A.

[redacted]

PART II. DESCRIPTION AND DISCUSSION OF DESIGN PROCEDURES AND METHODS

Contents

- -

2.1 Introduction

The blast pressure loading on the structures is a pressure-time function of the structures' size and shape, of the distance from the blast center, and of the explosive equivalent of the source of the blast impulse. These pressures were computed by means of the diffraction theory supplemented by model shock tube experiments.

Different sets of time-pressure curves [1] were furnished for different locations on different buildings of the test structure and these, of necessity, are used as exact values though an assumption of a ± 35% error in the intensity of the loading is considered in the design analysis. Other projects will require similar information and loads. Presumably this information will be available from the appropriate agencies in the near future.

2.2 Blast Pressure Loadings

2.2.1 Characteristics of the Shock Wave

The basic shock wave consists of an abrupt rise in pressure to peak positive intensity, a gradually decreasing positive phase, and a following negative or suction phase [2] of approximately twice the duration of the positive phase as shown in figure 2.2.1-1.

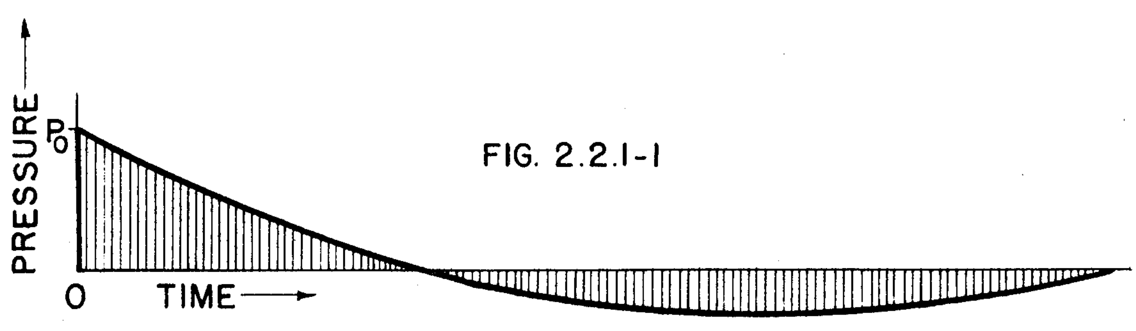


FIG. 2.2.1-1

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(1) C. W. Lampson, Pressure-Time Curves

(2) H. L. Bowman, "Bombs vs Buildings"

[redacted]

This pressure, or shock wave travels outward from the point of detonation at a speed which is approximately 1400 to 1500 feet per second at the location of the test structures. The pressure decreases in intensity but increases in duration with distance from the burst. As the shock wave reaches the face of an obstructing building, the wave is compressed and reflected producing pressures of increased intensity on the obstructing surface, the intensity depending on the shape, size and openness of the obstructing objects. The peak magnitude becomes approximately three times the side-on pressure though the duration of the positive phase remains the same. A typical curve showing the variation in pressure on the building face with respect to time is shown by figure 2.2.1-2.

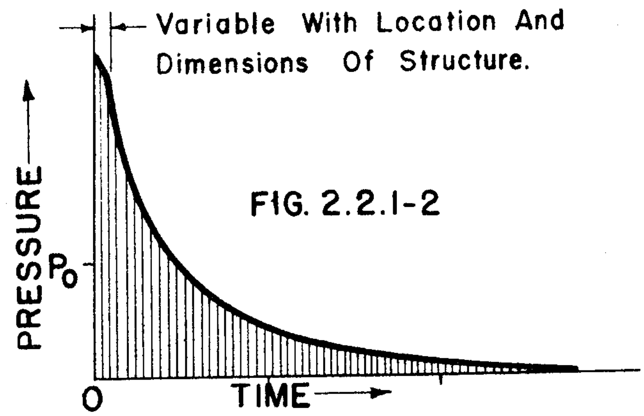

FIG. 2.2.1-2

During the time that the pressures are being reflected and built up on the front face, the shock front continues an uninterrupted advance over the top and around the ends of the structure, and finally reaches the rear of the building where pressures are exerted on the rear wall estimated to be as shown in figure 2.2.1-3. Local disturbances occur at the front edges of the sides and roof resulting in a varying of pressures along these surfaces as indicated by figure 2.2.1-4.

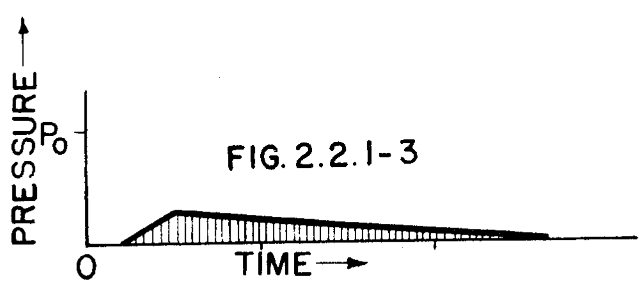

FIG. 2.2.1-3

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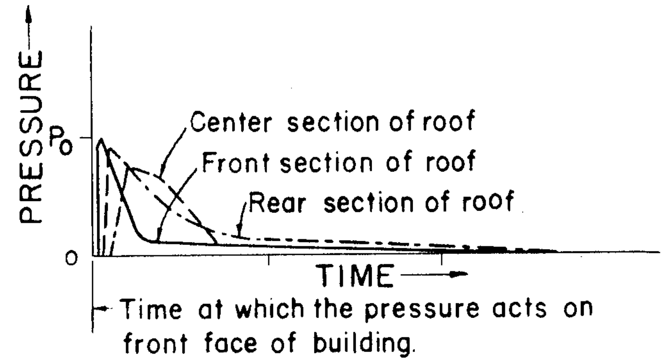


FIG. 2.2.1-4

The net translational force on the building including the forces acting on the front and rear walls is shown in figure 2.2.1-5.

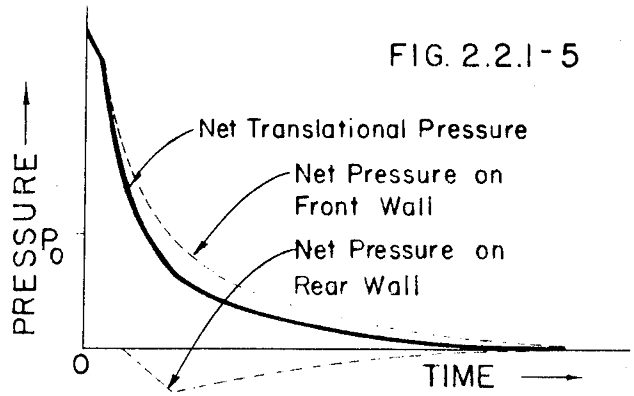


FIG. 2.2.1-5

Openings in the exposed face of the building not only reduce the total area subjected to the blast pressure but also permit the wave to enter the structure and back up the remaining loaded front face. The estimated pressure-time curves representing this loading condition are expected to be as shown in figure 2.2.1-6.

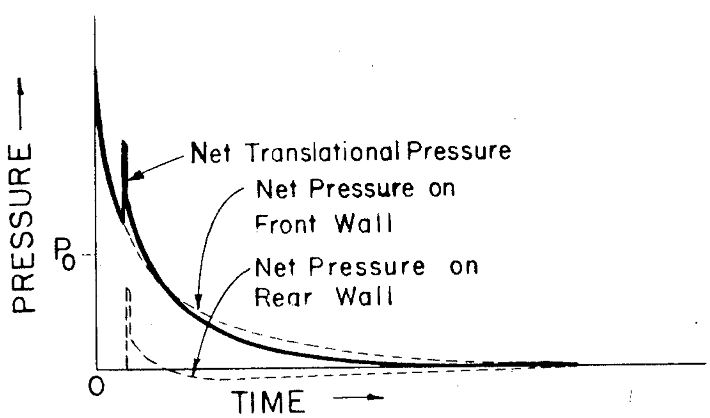


FIG. 2.2.1-6

[redacted]

The suction phase of the pressure loading is indicated by figure 2.2.1-1. During this suction phase the exterior pressure drops and the air within the closed buildings tends to force out the containing walls and slabs. Though this pressure is relatively small as compared to the positive reflected phase, it may be large compared to conventional loads and reinforcement, and structural details should be provided to resist this reversed loading.

2.2.2 Effect of Changes in Pressure Load Characteristics

Separate sets of pressure loading curves were provided in July 1949, October 1949, February 1950, March 1950 and April 1950 respectively. These loadings included the initial set; a second complete set with pressure increased approximately 33%; the third set with the pressures decreased to near the original peak loads but with a change in intensity-duration causing approximately a 50% increase in total impulse; a fourth set of pressure loads showing some increase in load duration and a material decrease in rear wall pressure and consequently a further increase in the net translational force; and finally a fifth set containing such modifications as proved necessary to obtain agreement between the theoretical pressures and the pressures indicated by extensive experimental studies being conducted at the present time.* These pressure curves are shown in Appendix 1. From this rather broad group of peak intensities, load durations, and impulse curves, investigated at 65%, 100% and 135% of the given intensity, certain general conclusions as to the effect of these loads and similar loads may be drawn which will be outlined briefly here:

(1) The behavior of the light brittle members acting as wall and roof panel units is dependent on the peak pressure and the increase in strength of the material with the rate of loading, and is independent of the load duration.

(2) The behavior of light ductile members acting as wall and roof panels is dependent on peak pressure, the increase in strength of the material with rate of loading, and on the ductility of the material, and is independent of the duration of the peak loading. Because these ductile members may be designed for deformation in the plastic range the required strengths of the members decrease by as much as 40% and 50% below that which would be required by design within the elastic range.

(3) Heavy ductile panels are governed by both the peak intensity of loading and the duration of the peak loading. The greater mass of the heavy panels acts favorably to limit the rate of acceleration and deformation and the maximum allowable deformation may be reached sufficiently after the peak intensity has passed to permit use of design strengths considerably less than the strength necessary to carry the peak intensity as a static load.

---

* Dr. Curtis W. Lampson, Head of Engineering Division, Ballistic Research Laboratory, Ordnance Division, U.S.A. and
Dr. Walker Bleakney, Physics Department, Princeton University, (Office of Naval Research Project).

[redacted]

(4) Building frames are sensitive not only to the peak intensity and the duration of the peak load itself, but also the total impulse and the time distribution of the impulse.

It is thus readily apparent that the separate elements of the building will react differently to the intensity and duration of the pressure loading and the designer of blast resistant buildings should know not only the possible errors in the loading intensity but also the variation in the shape of the pressure-time curve with distance in order to produce a balanced design.

It should be remembered that the members are being designed for a limit loading and large plastic deformations are assumed. This procedure, in certain cases, assumes considerable delicacy in design and a small addition to the design load may result in excessive deformations or failure of the member. For example, if the behavior of a light ductile material is considered, as shown below by figure 2.2.2-1 and as described in detail later in section 2.3.3, the difference between the ultimate strength and the load pulse may be only 10%. Therefore if the load is increased only 5% the differential between the resistance provided and the load will be halved and the deformation will be doubled. If the load increases 10% or more the member will fail.

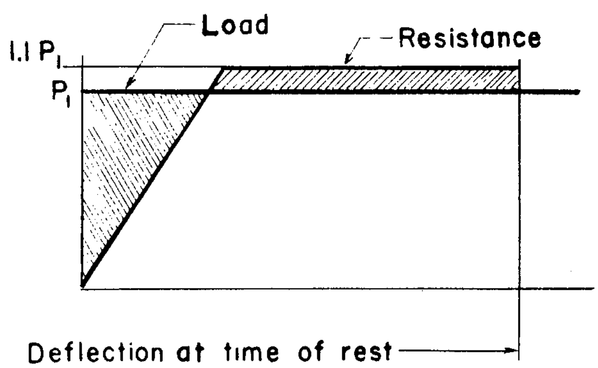


FIG.2.2.2-1

The overturning force of the net blast pressure on the front and rear walls is so large that the expense of providing mechanical anchorage against these loads is practically prohibitive. The blast pressure on the roof, however, serves as an important stabilizing agent against such motion. In order that full advantage may be taken of the developed roof pressures, recorded time-pressure curves obtained from the test buildings should be carefully studied and compared with the experimental and theoretical values.

A similar point of almost equal importance concerns the pressure on the footings projecting beyond the walls of the building. Deficiencies in righting moment resulting in either overturning motion or in an insufficient factor of safety against overturning, may often be overcome relatively inexpensively by extending the footing and using

[redacted]

the blast pressure on this area to anchor the building. In this case, as for the roof pressures, accurate values of the pressure intensity are important. Overestimating the pressure on the footing will result in a lowered factor of safety while an underestimate will result in excessive construction costs.

2.3 Behavior of Structures Resisting Static and Dynamic Loads

Contents

2.3.1 Introduction

Conventional structural design procedures deal with the application of static loads or of dynamic loads which are a relatively small part of the total load. Usually equivalent static loads are substituted for these dynamic loads. As the static loads are assumed to be of long duration, the stresses are kept well within the elastic limit to prevent continuing deformation and ultimate failure.

In contrast, structures designed to resist a single, short-time impulsive blast load, which will probably occur only once, may be limited only by deformations which will render the structure inadequate from a functional standpoint, or which will make it incapable of carrying the subsequent static loads.

Although equivalent static loads may be substituted for dynamic loads in conventional design, this procedure, while suitable for certain types of blast pressure framing, results in uneconomical designs in many other instances where full advantage might be taken of the short duration of the typical peak blast intensities, the mass of the yielding member, and the greater energy absorption available in the plastic range. In these cases the design should consider the mass, acceleration, deceleration, and vibration of the particular member under the intensity, duration, and shape of the given pressure curve. This condition is particularly true for high mass, slow acting members and frames.

[redacted]

The following few paragraphs of section 2.3 are added to describe, by means of simplified, idealized examples, the general concept of the design theory utilizing elastic and plastic resistance in carrying dynamic loads.

Throughout the description the applied force will be compared to a resistance developed in the structural member. While the resulting curves are conveniently simplified, it must be emphasized that the dynamic and not the static resistance is being shown, this dynamic resistance being a variable which is a function of the rate of loading. As an exaggerated example of this effect, if it is assumed that the dynamic strength is doubled under the rapidly applied load of the example illustrated by figure 2.3.3-1, the condition would then be such that the strength requirements for an instantly applied pulse would be doubled over the static load strength requirements but the available resisting strength of the member would also be doubled because of the assumed rate of loading effect. Therefore the original member which might be just adequate for the static load would also be adequate for the dynamic load. A discussion of the effect of the rate of loading on the dynamic strength will follow in section 2.4.2.

It is recognized that the unit stress will increase if the member is sufficiently strained to cause work hardening. This property is not shown in the assumed resistance curves used in the discussion immediately following because of the limiting values of strain adopted as the maximum permissible. Later portions of the report will discuss this property of the material including an estimate of the effect of the rate of loading on the initiation of strain hardening.

2.3.2 The Idealized Resistance Function

The resistance function for a member may be defined as the curve showing the relation between load and the displacement of the point loaded. For a simple tension member it would be the stress-strain curve. (For certain particular cases, described later, the resistance function may, as an alternate condition, be a curve showing the relation of the resistance with respect to time.)

Assuming the stress-strain curve as shown in figure 2.3.2-1 as representing the resistance function for a normal load applied gradually, the stored energy for strains within the elastic limit, ϵ_E, is completely transformed into potential energy of strain and can be recovered by unloading the member. The total stored energy per unit volume of the member at the elastic limit is

$$U_E = \frac{S_Y \epsilon_E}{2}$$

[redacted]

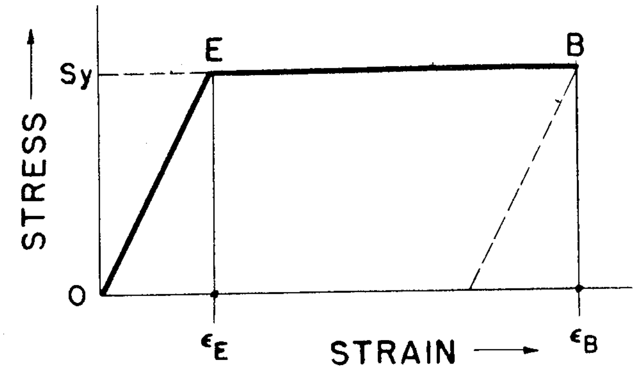


FIG. 2.3.2-1

If the load is maintained at the stress S_Y (figure 2.3.2-1), the element will continue to strain without further increase in stress. When the deformation reaches ϵ_B the added energy absorbed by the element will be

$$U_P = S_Y(\epsilon_B - \epsilon_E)$$

and the total energy will be the sum of U_E and U_P.

Assuming a unit volume of steel with an elastic limit of 30,000 p.s.i. and a modulus of elasticity of 30 x 10^6, deformed to total elongation of 2%:

$$U_E = \frac{30{,}000\text{ p.s.i.} \times 0.001''}{2} = 15\text{ in. lbs.}$$

$$U_P = 30{,}000\text{ p.s.i.} \times 0.019 = 570\text{ in. lbs.}$$

$$U_E + U_P = 585\text{ in. lbs.}$$

The energy absorbed during the 2% strain under the static loading is thus 39 times the energy absorbed in the elastic range. Assuming 2% strains to be permissible in plastic limit design, the energy absorbed in the elastic range therefore becomes relatively unimportant, amounting to only about 2½% of the total.

For very rapid loading, the shape of the resisting function is assumed similar; however, the yield strength and the work absorbed by the member will vary with the rate of loading.

It should be noted that the shape of the resistance function between zero deflection and the yield point (O and E on figure 2.3.2-1) appears as a straight line. As shown later, however, there seems good reason to believe that the shape of the curve within the elastic limit may vary from the linear stress-strain relationship, when rates of loading are high.

[redacted]

2.3.3 Absorption of Energy under Dynamic Loads

Assume a load P instantaneously applied and continuing with constant intensity applied to a member with a dynamic yield strength double the applied force as shown in figure 2.3.3-1.

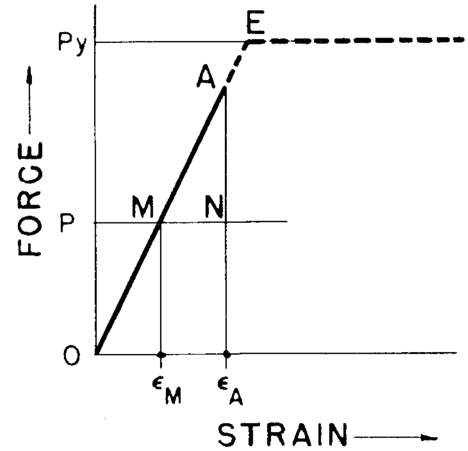


FIG. 2.3.3-1

At the time of initial loading the kinetic energy and the resistance of the member are zero and the force starting the mass in motion is P . As the loading continues the member will deflect, building up resistance. During this time the excess of the applied force P over the increasing resistance will be spent accelerating the mass of the member. The velocity of the member increases until the deformation reaches ϵ_M at which time the unbalanced force is zero but the mass of the element has obtained a peak velocity. At the time of the deformation ϵ_M the work done by the applied load is $P \times \epsilon_M$. Half of this energy is stored as potential energy in the element while the other half is transformed into the kinetic energy of the moving body, which continues to deform the element.

As the strain increases beyond point ϵ_M under the influence of the momentum, the resisting force becomes larger than the load P : and, due to the excess of resistance over the load, the velocity diminishes, becoming zero where the strain is ϵ_A .

Since the strain ϵ_A is within the elastic range at the time of zero velocity, $\epsilon_A = 2\epsilon_M$ and the potential energy stored within the element is $P \times \epsilon_A$, is twice the intensity and four times the potential energy which would be built up by a static load of the same intensity. To review this concept from the standpoint of work: the energy put into deformation, the product of force times motion $P \times \epsilon_A$, is represented on figure 2.3.3-1 by the rectangle $O P N \epsilon_A$. The energy absorbed by the deformation is represented by triangle $O A \epsilon_A$, the area of which must

[redacted]

equal the area of $OPN\epsilon_A$ when equilibrium is reached. As area $OMN\epsilon_A$ is common to both the rectangle and triangle $OA\epsilon_A$, it is apparent that triangle AMN will be equal to OPM . These triangles being similar, we see that the strain resulting from applying the load suddenly is twice what it would be under a gradually applied load, and that the stresses also are twice as high.

Using the same loading condition, but assuming a lower elastic limit as shown in figure 2.3.3-2, the velocity at the time that the deformation reaches ϵ_M , the point at which the net force is zero, is the same as described above.

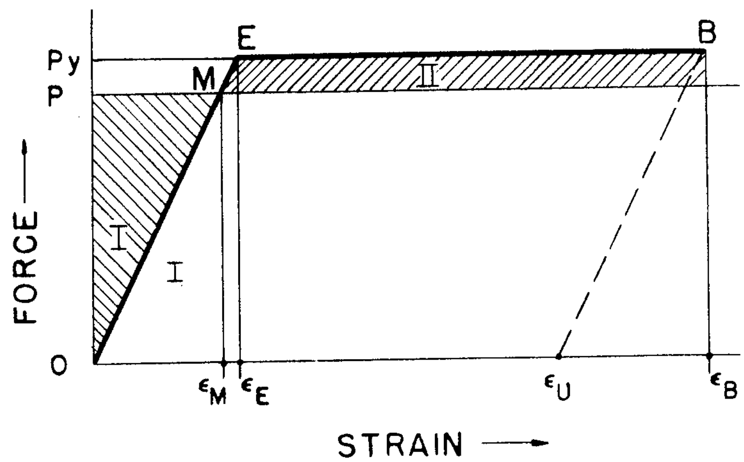


FIG. 2.3.3-2

Because of the energy acquired up to point M , the member will continue to strain with increasing resistance to point E , which represents the yield point, and with constant resistance thereafter.

As in the elastic case (figure 2.3.3-1), the velocity will become zero when area II becomes equal to area I, and the energy applied $P \times \epsilon_B$ will equal the energy absorbed, as represented by the area under the curve OEB

It is apparent that the final deformation ϵ_B depends on the magnitude of the yield strength P_Y . By proper design of the member, knowing the dynamic yield point of its material and intensity of loading, we may hold the deformation to any predetermined magnitude.

[redacted]

Using the same steel properties and 2% elongation listed in section 2.3.2 then:

$$\epsilon_B = 0.020$$

$$\frac{P \times 0.001}{2} = (P_Y - P)0.019 \text{ (Approximately)}$$

$$P_Y = 1.03P$$

After point B is reached the member will vibrate elastically about the deformed position under the load P until the oscillations are damped out. At this time the deflection will be the same as under the static load P.

If the load is then gradually decreased from point P, the rebound will parallel the original elastic rise and the unloaded strain or permanent set for the element at rest will be ϵ_U. The potential energy stored within the members is only 6% greater than the potential energy stored under a gradually applied load although the strain is 20 times as great. The balance of the applied energy will have been absorbed in plastic deformation.

The ratio P_Y/P may be called the dynamic load factor and indicates in this case that the dynamic yield strength of the member should be 1.03 times the applied load in order that the allowable deformation should not be exceeded. It should be noted that the dynamic load factor is the ratio of the equivalent static load, which would produce the maximum allowable deformation, to the actual dynamic load. This definition is also true when the strain does not exceed the elastic limit as shown in figure 2.3.3-1, where the dynamic load factor is 2.

So far only a constant and continuous pulse of long duration has been considered. The applied pulse may be irregular, may require a certain time to rise to peak value, and may die down completely. The effect of the time of rise of load is readily apparent from the preceding discussion where it was shown that an instantaneously applied force, (or zero time of rise) was twice as severe as a gradually applied load (a large time of rise).

For the effect of a load diedown assume that the pressure curve drops as shown in figure 2.3.3-3.

The member will have the same velocity at ϵ_M as was found for figure 2.3.3-1.

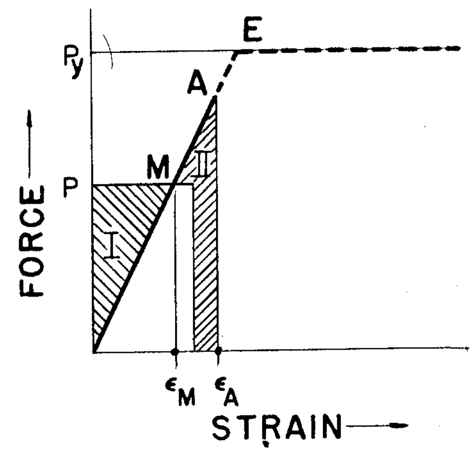


FIG. 2.3.3-3

The unbalanced energy would again be represented by Area I. Again the element will continue to strain until the absorbed energy (Area II), becomes equal to the applied energy (Area I). It is obvious that less strain and less stored potential energy is produced for this case than was the case for the continued pressure described in section 2.3.3.

If the duration of the applied dynamic load is less than the time required for the member to reach its yield point strain, the effect will be as shown in figure 2.3.3-4 and the applied load may be greater than the yield point resistance.

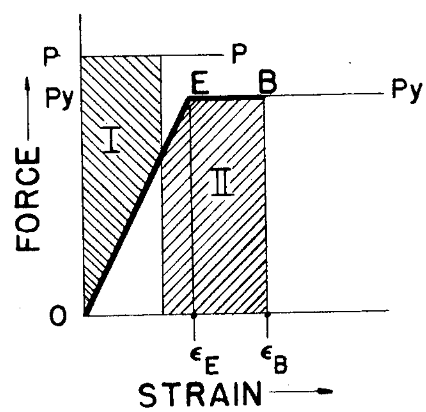


FIG. 2.3.3-4

Example: Assume the same steel properties and 2% elongation as in the previous examples but assume the particular loading duration shown in figure 2.3.3-5 instead of the long duration loading of the previous example.

$$P_Y(0.019) = (P - P_Y)0.001 + P_Y\frac{(0.001)}{2}$$

$$0.019P_Y = 0.001P - 0.0005P_Y$$

$$0.0195P_Y = 0.001P$$

$$P_Y = 0.051P$$

$$\text{Dynamic Load Factor } {P_Y}/{P} = 0.05$$

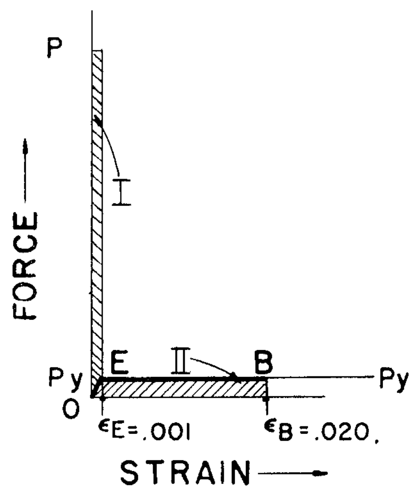


FIG. 2.3.3-5

The design strength needed for this example is only 5% of that which would be required if the load were static.

2.3.4 Design of Members in the Plastic Range Compared to Design within the Elastic Range

Assuming that a maximum strain of 2% is permissible in a member carrying a dynamic load of constant intensity in the plastic range, the examples of section 2.3.3 indicate that the dynamic load factor may vary from about 1.03 to less than 0.05 depending on the duration of the load in relation to the mass (rate of deforming) of the member.

[redacted]

This dynamic load factor gives the ratio of the required dynamic yield strength of the member to the load.

In the preceding illustrations the dynamic load factor was shown to be 2.0 if the strain is kept within the elastic range, may be 1.03 for a 2% deformation under an instantaneous load of constant intensity, and could be as low as 0.05 for a 2% deformation if the diedown is within the natural period of the member. It is evident that there is considerable economy in designing a member for as large a plastic strain as permissible.

Studies of light panels of small mass indicate that because of their quick response the typical blast pressure will act largely as an instantaneous constant pulse load and they should be designed for a factor slightly greater than 1.00 on the static strength if plastic deformation is permitted, and for a factor of 2.00 if the elastic limit is not to be exceeded. The static strength in every case, however, should be sufficient to carry the load existing at the longer durations.

Heavy monolithic panels, subjected to typical peak reflected pressure intensities of short duration compared to their particular rate of loading, may be designed in the plastic range for factors approaching the lower limit.

The absolute lower limit of design strength (0.05) is largely theoretical, however, as the duration and charactertistics of the typical A-bomb blast loads are such that it is more practical and economical to use thinner and stronger members than would be required to provide the necessary mass minimum strength panels.

2.3.5 Vibration under Dynamic Loads

Thus far, the loads and deformations have been considered only to the point where the motion of the member due to the initial unbalanced forces has been completely absorbed by the member through some type of deformation. The deflection at this time has reached a maximum, as indicated by point A on figure 2.3.3-1, and the member possesses stored energy which tends to return the member to its undeflected state, with a resultant vibration of the member. The amplitude of the vibration is determined by the unbalanced force, i.e. the difference between a theoretical force which would hold the member at its point of deflection and the actual force of the currently applied load; the duration of vibration effect will depend on the damping, or the rate of absorption of the energy.

The undamped vibration effect of a constant value, suddenly applied load of magnitude insufficient to cause deformations within the plastic range, is shown in figure 2.3.5-1.

[redacted]

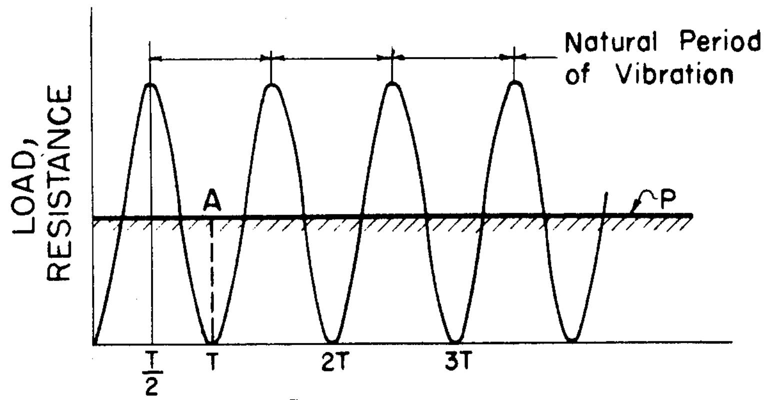


FIG. 2.3.5-1

Figure 2.3.5-2 indicates the same condition as described above except that the strength of the member is decreased so as to cause deformations in the plastic range. The amplitude of the undamped vibration in this case is much smaller than in the previous case.

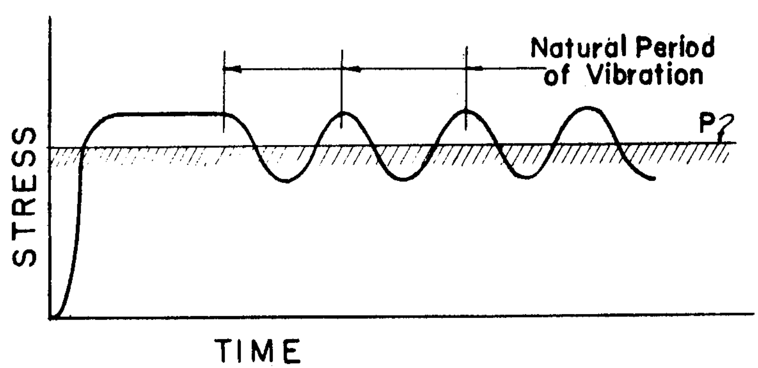


FIG. 2.3.5-2

If the loading has a time of rise, the unbalanced forces will be reduced. If the time of rise is slow enough, i.e. the rate at which the loading is applied is less than the minimum required to maintain equilibrium between the acting load and the resisting forces of elasticity, then there will be no vibration.

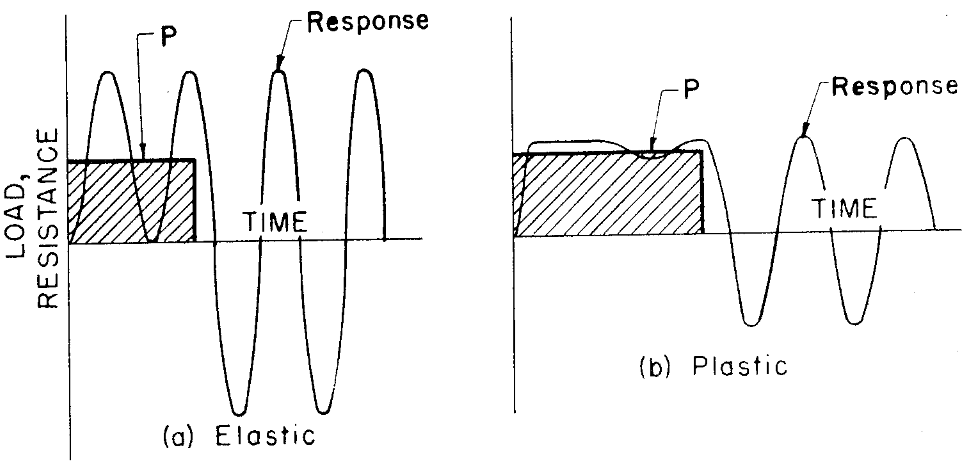
If the applied load dies down to zero, instead of remaining constant as previously assumed, at such a time that the stored energy is

[redacted]

just sufficient to return the member to its undeformed state, the vibration will end. For instance, in the elastic range shown on figure 2.3.5-1 if the load dies down instantaneously at time A when the stress is zero, no forces will exist to cause a second oscillation.

Although the phenomenon is not as easily expressed when considering stresses in the plastic range, it may be readily understood that the same action is affected if the load dies down at the proper instant or at the proper rate so that the stored energy is just sufficient to return the member to the point of zero deformation.

Both the instant when diedown begins, and the rate of diedown of the load, have important effects on the magnitude of the vibrations. From figure 2.3.5-3 (a) and (b) it is seen that diedown may possibly occur at such an instant that the amplitude of the vibration may become twice as large as would be the case if the diedown had not occurred. Other times and shapes of diedown will produce intermediate results somewhere between the complete stoppage of vibration, and the double amplitude case just described.



RESPONSE CURVES
ZERO TIME OF RISE — INSTANTANEOUS
DIEDOWN AT INSTANT OF MAXIMUM RESISTANCE

FIG. 2.3.5-3

Comparison of the above diagrams shows that in all cases vibration amplitudes can be appreciably reduced by designing members to be stressed into the plastic range.

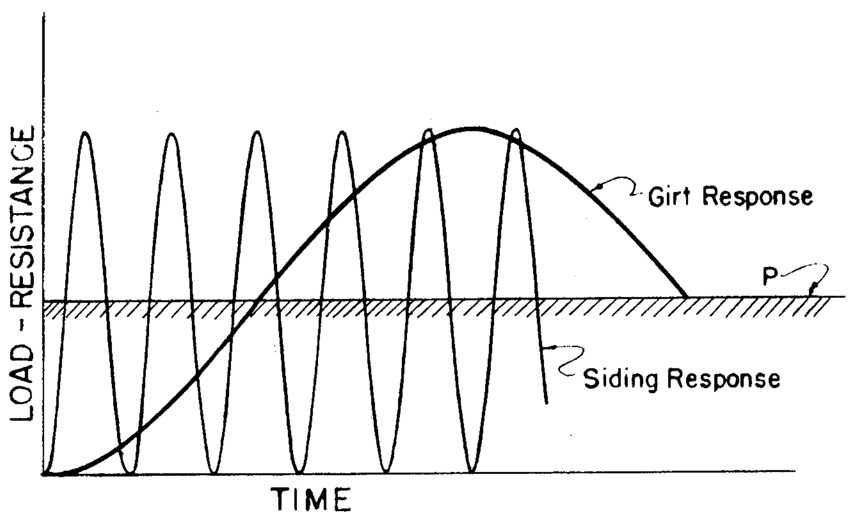
From a design standpoint, the most important aspect of vibration is that represented by figure 2.3.5-3. In most cases, the load applied to a structure will not diedown at a convenient instant but rather will cause reversal of stress and the members and member connections must be

designed for this condition. For example in designing quick acting panels under the effect of impulse loads having near instantaneous diedown characteristics, it may be impossible to determine, because of inaccuracies in the given loading and the adopted design theory, whether something less than full reversed strength may safely be provided.

If a constant load is suddenly applied and maintained for any time equal to or greater than one half the fundamental period of a member, the stress and deflection in a member acting at stresses below yield point value will be twice that due to a gradually applied load of equal magnitude. If the member is incapable of carrying this load elastically, a plastic deformation will take place. This plastic strain will be such as to completely dissipate the portion of the momentum of the system which is in excess of the elastic capacity of the supporting system.

If the load is gradually, rather than instantaneously, increased from zero to its maximum value and requires a time greater than four times the period of the member to reach its peak value, the vibration will be negligible and the resultant deflection can be computed from the usual static conditions. As the time of rise of the stress decreases, the maximum deflection will increase approaching the condition described for a suddenly applied load.

When a member having a high natural frequency is supported by one having a much lower frequency, as in the case of V-beam siding supported on heavy steel girts, vibrations of the siding have negligible influence on the effective load transferred to the girt. This follows from the fact that a pulse whose duration is short relative to the fundamental period of the resisting member has only a very small effect on the deflection of the member. Since the average of the pulses is the applied load, the conclusion is that the load transmitted to the girts is equivalent to the load applied to the siding. As shown in figure 2.3.5-4, the girt response to this applied load will be twice what it would be if load were gradually applied.



· FIG. 2.3.5-4

Going one step further, it can be concluded that the load transmitted to the frame will be equal to the load applied to the siding minus a small percentage due to an energy loss which appears as damping of the siding and girt vibrations.

From the previous discussion it is apparent that for the condition without damping and with a constant load suddenly applied, the stress of a member oscillates between either zero or some value greater than zero, and the maximum stress reached under the dynamic load. The reactions may be expected to vary similarly. Thus we see that so long as the load is maintained constant, the direction of the reactions will remain opposite to the direction of the applied loads, whether the damping effect occurs or not.

If instead of remaining constant, the load is suddenly removed at a time when the deflection is maximum and the velocity is zero, a reverse deflection of equal magnitude will occur. For a member acting entirely within the elastic range the reactions will now be not only twice the magnitude of what they would be under a gradually applied load, but they will reverse in direction during each period of vibration.

When a member is not strong enough to carry the load in the elastic range, vibration will not begin until energy is absorbed by deflection in the plastic range, as shown on figure 2.3.5-2, and the amplitude of the vibrations while the load continues will be determined by the difference between the yield point stress and the stress which would exist if the load had been gradually applied. This may be expressed numerically as follows: If a statically loaded member carried a stress S, and its yield point is $1.1S$, the same load applied suddenly will bring the member's stress to $1.1S$, and there will be a plastic deformation, until the initial momentum is absorbed. Then vibration will begin such that the stress will fluctuate from $1.1S$ to $0.9S$ in cycles corresponding to the natural period of vibration of the member.

If in the same case, the load is suddenly removed, again at a time when stress and deflection are maximum, vibrations will be such that the stresses will oscillate from $+1.1S$ to $-1.1S$ and the reactions of the member will reverse in direction in tune with the natural period of vibration of the member.

[redacted]

For the condition of a sudden but incomplete drop-off of the load, the effect is similar to that discussed above and is illustrated in figure 2.3.5-5 below

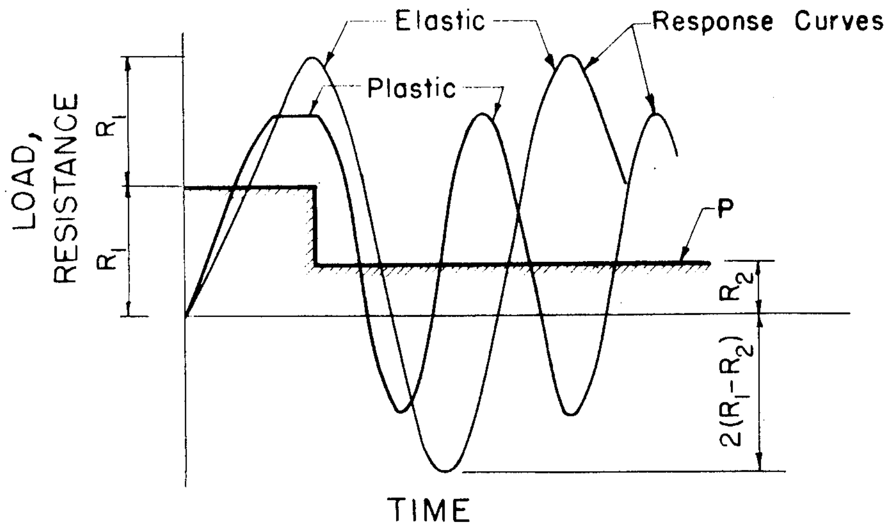


FIG. 2.3.5-5

Where the load curve has a uniform shape the vibrations will take place about the corresponding static deflection curve until they are dissipated by damping as shown in figure 2.3.5-6.

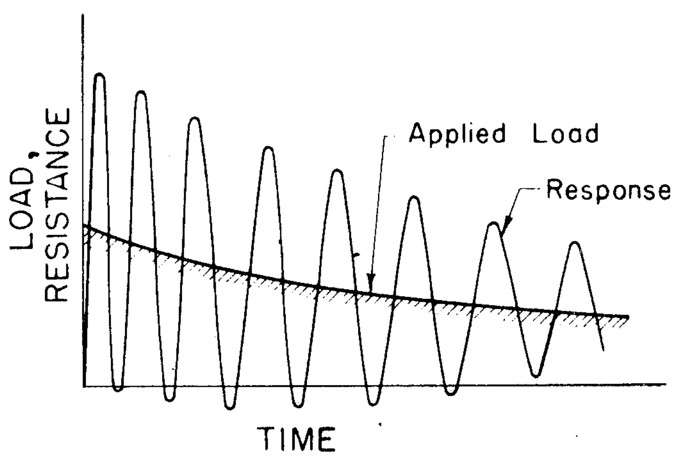


FIG. 2.3.5-6

In determining the most critical requirements for anchoring a member to its support, we see that the following conditions will give the worse case: (a) The member is subjected to an instantaneous application of load; (b) the load is instantaneously removed at the peak of the first vibration when the potential energy of the member is a maximum, and before damping has occurred; (c) the member is of sufficient strength so that stresses remain within the elastic limit. This condition was assumed in the design of the test structure members whenever the data on the applied loads indicated a sharp drop-off in the design load curve. When the design curve had a gradual shape similar to the curve shown in figure 2.3.5-6, the maximum possible rebound reaction was found to be less than 25% in the presence of nominal damping.

2.4 Behavior of Materials and Members under Static and Dynamic Loads

Contents

- -

2.4.1 Characteristic Behavior of Materials under Stress

Numerous materials are used in conventional construction and presumably can be used with more or less success in blast resistant buildings. Such materials may be classified into definite groups and may be expected to behave in accordance with the characteristics of their particular group.

As shown earlier in the discussion of strain energy, ductile materials appear most promising as blast resisting materials and the combination of ductility and mass is of even greater advantage.

Brittle materials permit only small deformations beyond the elastic limit and will therefore fail suddenly. They are generally of little value for use in blast resistant structures as they must be designed for the high dynamic load factor required by deformations limited to the elastic range. Unless substantially cheaper in cost per unit of strength, which is not generally the case, the brittle materials seem of value only in curtain walls where failure may be acceptable.

Reinforced concrete offers the decided advantage of combining the ductility of tensile steel with the mass of concrete.

Tough materials which can withstand heavy shocks and absorb large amounts of energy posses the desirable characteristics required. Toughness depends on strength and ductility. Judging from the blast resistance shown by various structures in Japan, reinforced concrete and structural steel may be considered tough; brick and block units, in general, are not tough.

Materials which are combustible or sensitive to heat, such as unfireproofed steel, are of limited value regardless of inherent strength and toughness

2.4.2 Stress-Strain Relationship of Concrete and Steel Under Static and Dynamic Loading

A. Tensile Tests of Steel

It will be assumed that for static loads the stress-strain curve of steel used in building construction is as shown in figure 2.4.2-1.

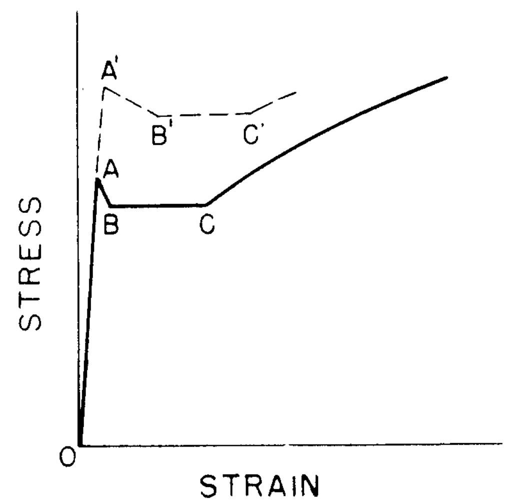


FIG. 2.4.2-1

From O to A the stress is proportional to the strain with a modulus of elasticity of 29 to 30 x 10^6 pounds per square inch. For strains up to point A the material will fully recover its original form upon removal of the load. After point A , the upper or apparent elastic limit, there may be an immediate drop in stress to point B , beyond which the stress remains constant while the member continues to strain to point C . For strains beyond point C the stress will increase again as a result of strain hardening.

[redacted]

Under static loading strains of $1\frac{1}{2}\%$ to 2% may be expected before the initiation of work-hardening whereas under high rates of loading this action seems to occur at strains approaching 3% to 4%, depending on the rate of loading. The assumption of an idealized resistance function such as shown in figure 2.3.2-1 thus seems reasonable and adequate for expressing work absorption in a majority of the members. Certain special cases of greater strains and the discussion of the effect of strain hardening will be presented later.

Most of the test data was obtained using structural grades of steel. It is assumed in the design of the test structures that the harder intermediate grades will behave in a similar manner, at least for the smaller strains. Additional dynamic tests may be necessary on this material.

As mentioned previously, the rate of loading has a pronounced effect on the stress-strain curve. According to Johnson[3] the yield point is considerably affected by speeds of the testing head over approximately 0.025 inches per minute. In 1931 Nadai[4] listed conclusions by Ludwik, Cassebaum, and Bailey to the effect that the dynamic stress will probably vary with velocity of strain in accordance with a logarithmic law.

In 1942 the National Advisory Committee for Aeronautics published the results of tension tests on 16 metals and alloys under dynamic loading with the velocity of deformation held constant in each of the tests.[5] Specimens of hot rolled steel (SAE 1020) tested under static loading and at rates of 15, 35, 60, 90, 120 and 150 feet per second, exhibited stress-strain properties as shown in figure 2.4.2-2.

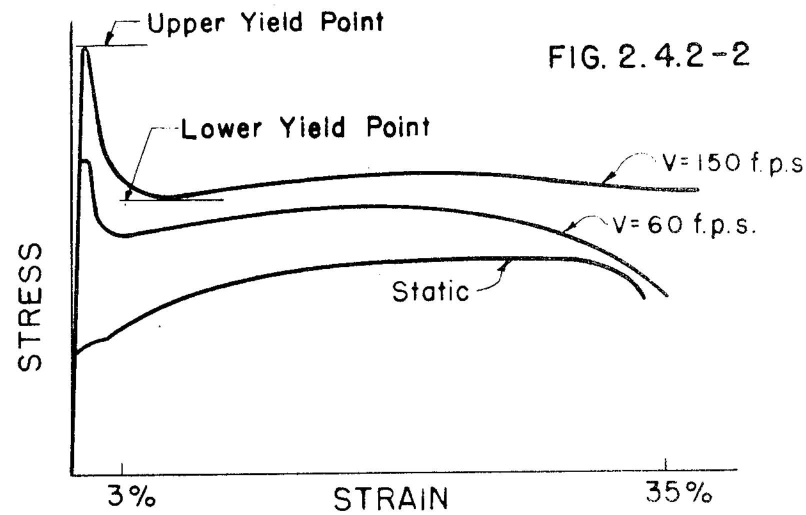


FIG. 2.4.2-2

(3) M. O. Withey, J. Aston, F. E. Turneaure, Johnsons Materials of Construction
(4) A. Nadai, Plasticity
(5) D. S. Clark, "The Influence of Impact Velocity on the Tensile Characteristics of Some Aircraft Metals and Alloys."

[redacted]

Clark in a 1949 American Society for Testing Materials report (6) described further dynamic tensile tests as conducted at California Institute of Technology in which the time to reach the yield point t_{yp} and the time to initiate plastic strain t_{pl} were observed. In these tests, loads varying from 37 k.s.i. to 50 k.s.i. were applied to annealed 0.19% carbon steel specimens at an initial rate of loading of 5000 k.s.i. per second. A typical stress-strain-time relationship for this loading is shown in figure 2.4.2-3.

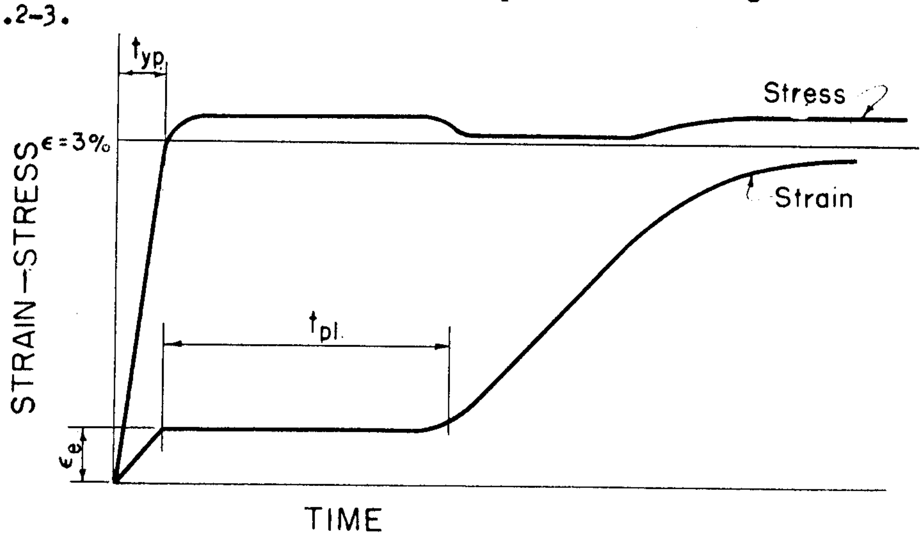


FIG. 2.4.2-3

Similar tests (7) were also conducted at Massachusetts Institute of Technology in 1949 on structural grade reinforcing steel using the same equipment as used in the California Institute of Technology tests. However, the initial rate of loading was varied from 1200 k.s.i. per second while the maximum load varied from about 50 to 75 k.s.i.

Qualitative information and some approximate quantitative values may be drawn from these tests. All indicate that the elastic modulus is not measurably changed, while the yield point is raised by rapid loading. While the Massachusetts Institute of Technology and California Institute of Technology tests were limited to a 3% elongation, the National Advisory Committee for Aeronautics tests were conducted to failure of the specimens and showed an increase in ultimate strength and total work absorbed with the high rates of loading. The National Advisory Committee for Aeronautics tests also showed that the lower yield point is generally not reached until 3% strains are attained. The Massachusetts Institute of Technology and California Institute of Technology tests show a drop in stress below the upper yield point at strains less than 3% but attribute this drop to

(6) D. S. Clark and D. S. Wood, "The Time Delay for the Initiation of Plastic Deformation at Rapidly Applied Constant Stress."

(7) J. B. Wilbur, R. J. Hansen and K. Steyn, "Behavior of Reinforced Concrete Structural Elements under Long Duration Impulsive Loads."

[redacted]

the characteristics of the machine as well as the properties of the material. At the highest rates of loading, these tests indicate that the upper yield point is maintained over a greater elongation suggesting that characteristics similar to those observed by the National Advisory Committee for Aeronautics might have been attained if the rate of loading were as high. Work-hardening was not apparent in any of the tests for strains below 2%-3%.

CHANGE IN PHYSICAL PROPERTIES OF STEEL

RAPID RATES OF LOADING COMPARED TO STATIC LOADING

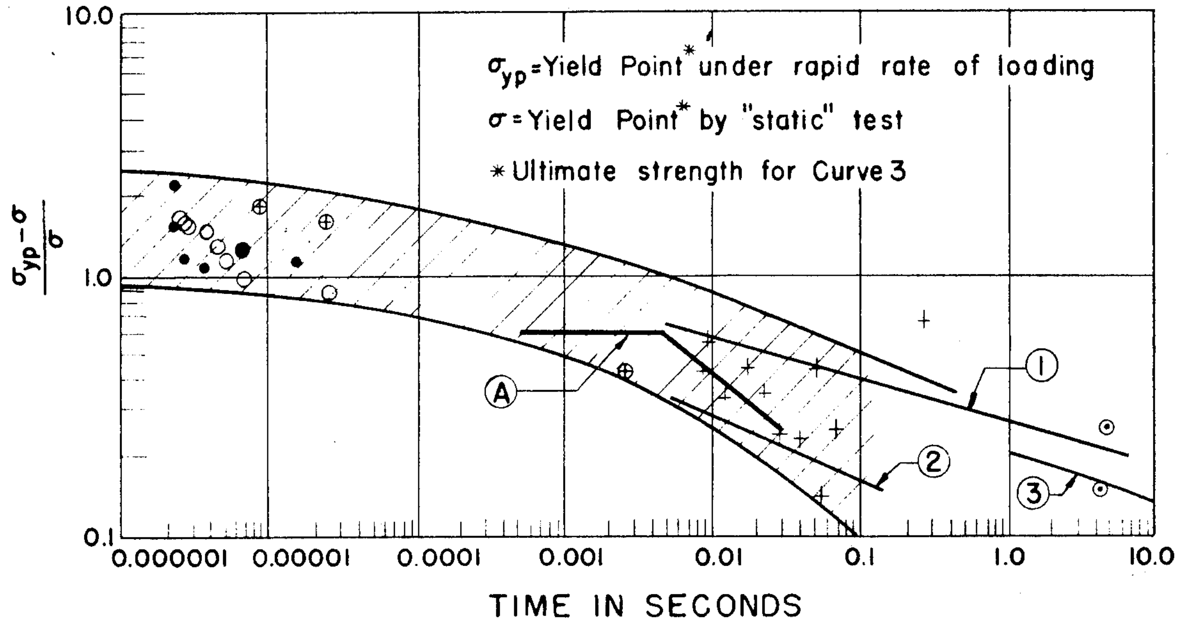
SPECIMENS	Av. Rate of Loading k.s.i./sec.	% CHANGE				
		Total Energy	Energy $\epsilon = 3\%$	Yield Point	Ultimate Strength	Strain at Rupture
S.A.E. 1020 (HOT ROLLED)	21.6×10^6	-10	+84	+107	+15	-10
	54.0×10^6	+28	+142	+216	+32	-11
S.A.E. 1035 (ANNEALED)	3.6×10^6	+22	+169	+84	+25	-2
	43.2×10^6	+43	+136	+151	+46	+4
*REINFORCING STEEL (STRUCTURAL GRADE)	448		+15	+17		
	904		+20	+26		
	1100		+12	+16		
	1360		+23	+42		
	1475		+20	+23		
	2220		+20	+25		
	2930		+22	+34		
	4050		+25	+43		
	5310		+20	+33		
	7640		+34	+43		
	8330		+30	+56		

* Tests performed to about 3% Strain

FIG. 2.4.2-4

The available test data is summarized in the table of figure 2.4.2-4 and figure 2.4.2-5. Although these results are scattered, a trend toward increased resistance with high rates of loading is indicated, and in the absence of more specific data the shaded area of figure 2.4.2-5 shows the probable range in yield points to be expected. From the table of figure 2.4.2-4

VARIATION IN PHYSICAL PROPERTIES
WITH RATE OF LOADING



TIME TO REACH YIELD POINT - ALL POINTS AND SHADED AREA
TIME TO INITIATE PLASTIC STRAIN - CURVES (1) & (2)
TIME TO REACH ULTIMATE STRENGH - CURVE (3)

LEGEND

- ● S.A.E. 1020, Hot rolled[5]
- ○ S.A.E. 1035, Annealed[5]
- + Reinforcing steel, Structural grade[7]
- ⊕ Low carbon annealed steel[8]
- ⊙ Mild steel[9]

(1) 0.19 % Carbon annealed steel[6]
(2) Reinforcing steel, Structural grade[7]
(3) 6" x 12" Concrete Cylinders[10]
(A) Empirical recommended curve

FIG. 2.4.2 -5

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(5) D. S. Clark, "The Influence of Impact Velocity on the Tensile Characteristics of Some Aircraft Metals & Alloys."

(6) D. S. Clark and D. S. Wood, "The Time Delay for the Initiation of Plastic Deformation at Rapidly Applied Constant Stress."

(7) J. B. Wilbur, R.J. Hansen, and K. Steyn, "Behavior of Reinforced Concrete Structural Elements under Long Duration Impulsive Loads."

(8) M. J. Manjoine, "Influence of Rate of Strain and Temperature on Yield Stresses of Mild Steel."

(9) E. A. Davis, "The Effect of the Speed of Stretching and the Rate of Loading on the Yielding of Mild Steel."

(10) P.G. Jones and F. E. Richart, "The Effect of Testing Speed on Strength and Elastic Properties of Concrete."

[redacted]

it can be seen that the ultimate strength and strain energy is increased with rapid loading while ductility undergoes no appreciable change. Pending observation of the actual behavior of the test structures and possibly some additional experimental laboratory data, as may be shown necessary from the performance of the test structure, the following design procedure is recommended.

From curve A (figure 2.4.2-5), which represents the recommended increased yield point properties, a trial resistance may be chosen consistent with the natural period of the element. T_{yp} will be approximately $T/4$ for a disturbance with no time of rise and $t_0 + T/4$ for a disturbance with a time of rise equal to t_0 if the member is designed for large plastic strains. Upon analysis by the step-by-step impulse procedure an actual time, t_{yp}, (figure 2.4.2-6) will be found. This value of t_{yp} must yield a resistance from curve A which checks the value used in the computation. Because of the greater uncertainty involved in the analysis of very quick acting elements and also because there is only a very limited amount of test data available, it is recommended that 60% be taken as the upper limit for increase in yield point, as indicated by curve A .

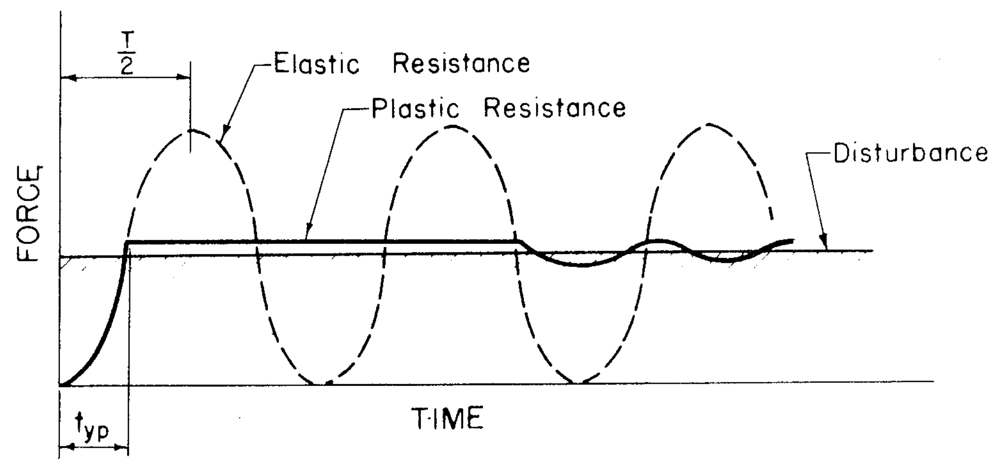


FIG. 2.4.2-6

The resisting functions used in the design analysis of the test structures were arrived at as follows:

(1) Concrete Walls, Slabs and Beams

The resistance functions for the concrete members, which may be expected to be influenced by cracking and slip of the reinforcing bars, were derived directly from the Massachusetts Institute of Technology laboratory beam tests. Curve A was used only for comparison and check.

(2) Light Gage and Structural Steel Wall Elements

The increase in strength for the rapidly loaded steel wall elements was estimated directly from Curve A .

[redacted]

(3) Steel and Concrete Frames

The slower moving frames were designed for a 25% increase in strength in accordance with the recommendations adopted by agreement between the Corps of Engineers, the Government consultants and the Contractor. This agrees with the value indicated by curve A .

B. Compression Tests of Steel and Concrete

Plastic materials, such as mild steel, may be expected to have the same static yield point, and similar increases in yield point and energy absorption under high rates of compression loading as are exhibited under tension loading. The ultimate strength under compression loading is unimportant as the material will spread and continue to carry load under large strains, the usefulness of the member being controlled by maximum permissible deformations or by buckling stability rather than by failure of the material itself.

The important characteristic of brittle materials, such as concrete, is that the strain at failure is too small to absorb much energy. For this reason, reinforced concrete members have been purposely under-reinforced so that the reinforcing steel would reach its yield point and undergo considerable elongation before the member fails.

Concrete, like steel, shows increased strength under short time loads. Experiments by the National Bureau of Standards, (11) as quoted by Dr. Hansen, show that concrete cylinders subjected to dynamic loading have approximately 10% greater strength than is obtained from standard tests. The loading for these tests had a substantial time of rise.

For the purpose of the proposed design analysis, the information provided by F. E. Richart (10) in tests made by loading 6" x 12" cylinders to failure at rates of loading varying from 3870 to 0.12 p.s.i. per second are of the greatest direct use. From these tests it was found that the increase in strength with rate of loading was represented fairly well by the following relationship:

$$S = S_1 (1 + k \, \mathrm{Log}_{10} R)$$

where

S = Strength at the given rate of loading

R = Rate of stress application in p.s.i. per second

S_1 = Strength at rate of stress application of 1 p.s.i. per second

k = a constant having a value of about 0.07 for 7-day tests and 0.08 for 28-day tests

(10) P. G. Jones and F. E. Richart, "The Effect of Testing Speed on Strength and Elastic Properties of Concrete."

(11) D. Watstein, "Investigation of Properties of Plain Concrete Under Impact."

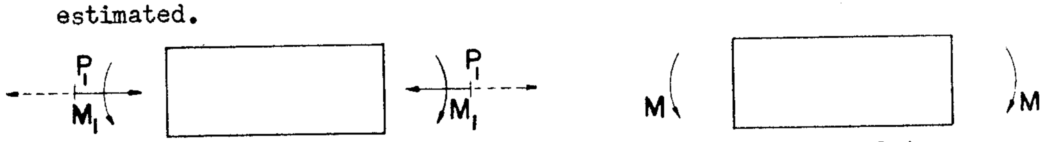
[redacted]

The curve defined by this equation is plotted on figure 2.4.2-5 with the results of the rapid loading rates on steel specimens. If the rates of loading experienced in the blast resistant structures are substituted into the above equation the result indicates an increase in strength of 47%, for the more rapidly loaded elements and 32% for the slower moving elements and frames.

The increased concrete strength under the higher rates of loading has little effect on members subjected to bending alone, is somewhat more important for members under combined bending and axial load, and is quite important for members with loads of small eccentricity. However, as the lateral loads largely control the member size of the columns and other members which have axial loads and because of the desirability of preventing abrupt column failures at loads of smaller eccentricities, the increase in concrete strength was neglected in the design of the test structures.

2.4.3 Members under Static and Dynamic Bending Loads - with and without Axial Loads

Given the above characteristics of materials in tension and compression, the behavior of members subjected to bending moments and with (2.4.3-1,a) or without (2.4.3-1,b) axial forces may be readily estimated.

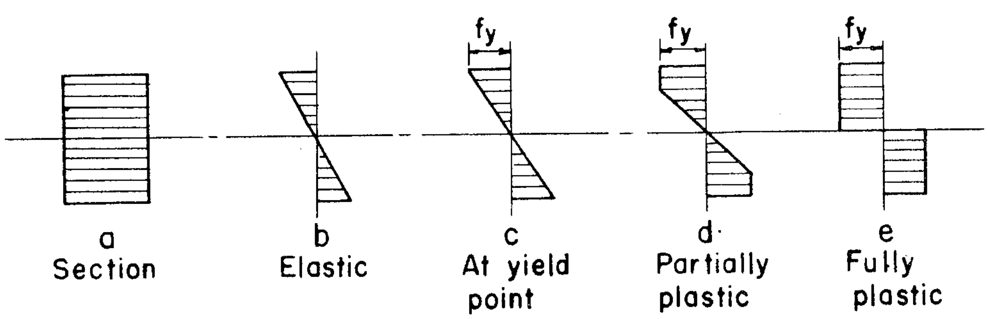


Moment and Axial Load (a) Moment Only (b)

FIG. 2.4.3-1

A. Rectangular Sections - Static Loads

Figure 2.4.3-2 shows the behavior of a rectangular beam without axial load and subjected to a progressively increasing bending moment. At first the stresses are within the elastic range (b) and as the load increases the stress-strain function remains linear until the extreme fibre reaches the elastic yield point (c). As the strain increases beyond this point, the extreme fibre stress will remain constant while yield point stresses approach nearer to the neutral axis (d) finally reaching the condition where the section is almost entirely plastic (e).



RECTANGULAR SECTION-BENDING ONLY

FIG. 2.4.3-2

[redacted]

The ultimate bending resistance for a rectangular beam under full plastic bending will approach 150% of the maximum bending resistance when all fibres are within the elastic range. See figure 2.4.3-3 a.

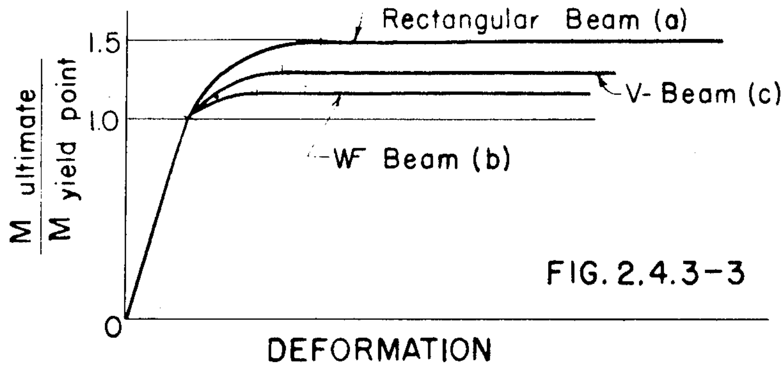


FIG. 2.4.3-3

B. Wide Flange Members - Static Loads

Wide flange sections act in a manner similar to the rectangular section shown in figure 2.4.3-2 except that the increase in moment from the time the extreme fibres reach yield point stress (c) to the condition of full plastic bending (e) is small because of the small additional sectional area available in the relatively narrow web (figure 2.4.3-3 b).

A study of the basic flexural behavior of wide flange beams loaded up to and beyond the yield point with extensive exploration of the accompanying elastic and plastic strains was presented by Luxion and Johnston (12) in 1948. This report confirms, in general, the design procedure outlined for the rectangular beams. The report also shows that for all currently rolled structural shapes listed in the American Institute of Steel Construction Handbook, residual stresses and plastic buckling will not materially reduce the expected design strengths.

The following study of a fixed end, wide flange, cantilever member (see figure 2.4.3-4) describes the formation of the plastic hinges.

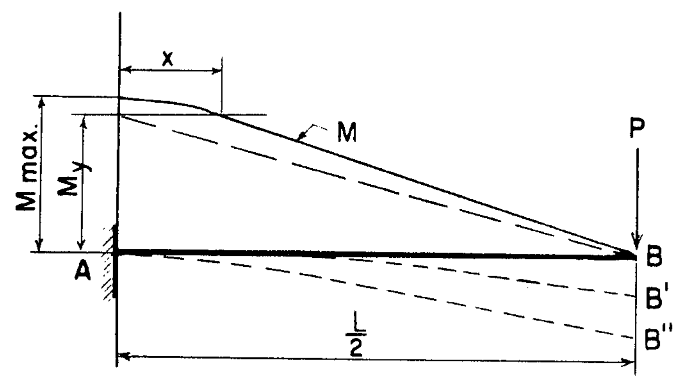


FIG. 2.4.3-4

(12) W. W. Luxion and B. G. Johnston, "Plastic Behavior of Wide Flange Beams."

As the load P is applied the moment at A will increase until the stresses in the extreme fibre reach the yield point. Within this elastic range the curvature in the member will be proportional to M/EI and the deflection at this time will be as shown for B'

As the load is increased a plastic resisting moment is developed at the support, the maximum value being from 10% to 15% greater than the yield point resisting moment. With the increase in moment at the support, the moments along the entire member will be increased proportionally and the yield point in the extreme fibres will be reached at a point a distance x from the support. The angular change necessary to develop this full plastic moment may be expected to be approximately 4 to 5×10^4 radians per inch for 8" wide flange members [12] and, assuming a linear variation of strain from the neutral axis, the elongation of the extreme fibres will be approximately 0.2%.

If the idealized stress-strain curve of figure 2.3.2-1 is assumed correct, $M_{max.}$ will represent the ultimate plastic moment regardless of continued strain. Under this assumption, however, the bulk of the rotation would be limited to a narrow strip immediately adjacent to the support and will result in large strains in this region.

It seems reasonable that large strains will not occur at the support without the initiation of work-hardening. The yield point will then move further away from the support, the exact length of the plastic area depending on the rate of increase in stress with strain hardening and the geometry of the moment diagram. In the design of the test structures the expected increases in moment capacity due to strain hardening was not considered because available data do not indicate that this behavior will be an important factor for the strains advocated in this report.

C. Light Gage Sections - Static Loads

Stress distribution in light gage sections will be similar to that in the wide flange sections except that the stress in the material may be limited to less than yield point stresses if local instability controls the section. Methods for determining the elastic stability of a section are too involved to list here but may be obtained from standard texts (13,14,15,16,17,18) and from research papers. If the flange and web are stable the resistance curve for a corrugated light gage section would be approximately as shown in figure 2.4.3-3,c.

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(12) W. W. Luxion and B. G. Johnston, "Plastic Behavior of Wide Flange Beams",
(13) S. Timoshenko, Theory of Elastic Stability
(14) E. E. Sechler & L. G. Dunn, Airplane Structural Analysis
(15) J. E. Younger, Mechanics of Aircraft Structures
(16) Light Gage Steel Design Manual, American Iron and Steel Institute
(17) G. Winter, "Strength of Thin Compression Flanges"
(18) G. Winter, and R. H. J. Pian, "Crushing Strength of Thin Steel"

[redacted]

D. Idealized Bending Resistance Function - Steel Members under Static Loads

The formation of the idealized resistance function as shown in figure 2.4.3-5 neglects the rounding of the curve immediately beyond the elastic yield point as shown in figure 2.4.3-3.

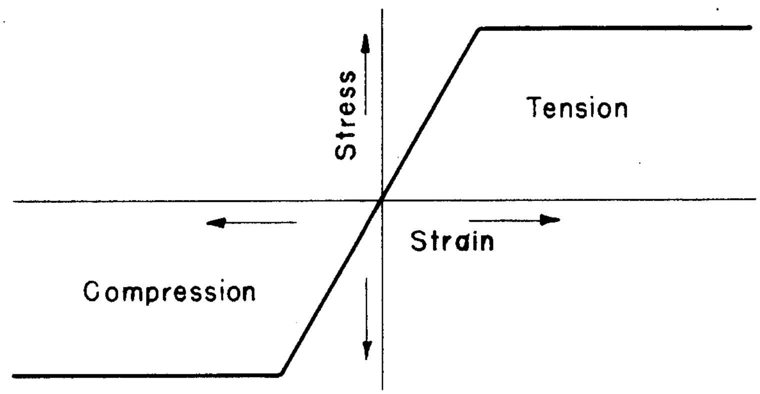


FIG. 2.4.3-5

Assuming a total strain of 2% this assumption would result in an error of total work absorbed of less than 2% for the wide flange, light gage and rectangular sections.

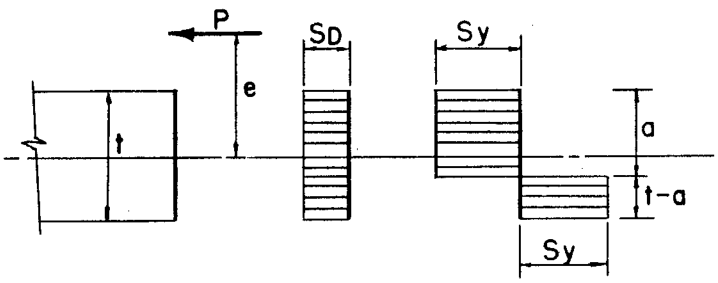
E. Steel Members under Combined Bending and Direct Stress - Static Load

Assuming that for bending in the plastic range, plane sections of the member continue to remain plane and that the longitudinal stress in each fibre continues to increase until the yield point is reached and thereafter remains constant in accordance with the assumed stress-strain curve, the stress condition for any combination of axial load and bending-moment may be easily found. However, as the combinations of axial load and bending-moment producing full plastic stress are more important for the purpose of this investigation than lesser loads, the discussion will be limited to the condition of ultimate strength.

The structural frames subjected to blast-pressures will be at rest when the axial load is first applied. Then, while still subjected to a variable axial load, the members will begin to deform causing a continuously changing load eccentricity. To facilitate design, under these varying conditions of bending and direct load, the simplest procedure is to plot a single curve showing the computed ultimate moment capacity for various values of direct stress. The procedure is as follows:

[redacted]

Consider a homogeneous rectangular steel beam of unit width subjected to an axial load, P and a bending-moment, M (equal to $P \cdot e$), as shown in figure 2.4.3-6.



(a) (b) Axial Load (c) Combined Load

COMBINED BENDING AND AXIAL LOAD
(Fully Plastic Condition)
FIG. 2.4.3-6[19]

Let P = Axial Load = $S_D \cdot t$

and M_P = Maximum Moment Of The Fully Plastic Member = $P \cdot e$

then $M_P = S_Y \cdot a(t-a)$

$$P = S_D \cdot t = S_Y(2a-t)$$

and $$M_P = \frac{t^2}{4S_Y}\left(S_Y^2 - S_D^2\right)$$

For Bending Without Axial Load

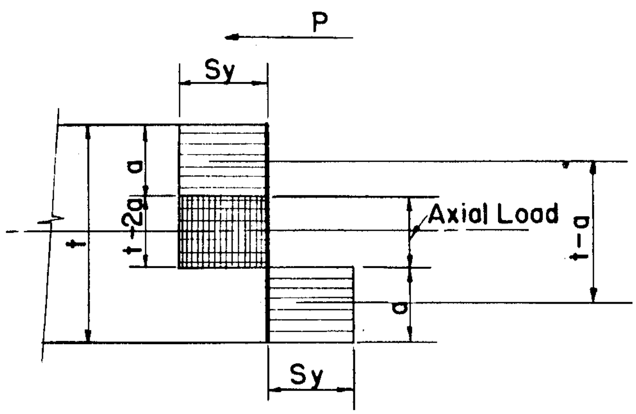
$$M_P = \frac{t^2 \cdot S_Y}{4}$$

(19) J. F. Baker, " Review of Recent Investigations into the Behavior of Steel Beams in the Plastic Range. "

[redacted]

In many cases, particularly for members other than rectangular sections the moment-axial load relationship for ultimate loadings may be more advantageously determined as follows:

Assume the same rectangular beam with notations as shown in figure 2.4.3-7.



Sy = Yield point stress
$Ac = Sy \times a$
Ac' = Section carrying Axial Load
$= Sy(t-2a)$
$A_T = Sy \times a$
M_p = Moment at fully plastic section

FIG. 2.4.3-7

Consider the axial load as carried on the area nearest to the neutral axis:

$$P_A = (t-2a)\,S_y$$

The remainder of the section will form the couple representing the maximum moment capacity for the given axial load:

$$M = S_y \cdot a(t-a)$$

For any given member various axial loads may be assumed, the area required for the axial load subtracted, and the net resisting moment for the fully plastic section may be found by multiplying the arm between the centroids of the remaining areas by the product of the remaining area and the yield point stress.

[redacted]

This solution will give the same result as obtained in the previous derivation:

$$P = S_y(t-2a) = S_y \cdot t - 2a S_y = S_D \cdot t$$

$$a = -\frac{(S_D \cdot t - S_y \cdot t)}{2 S_y}$$

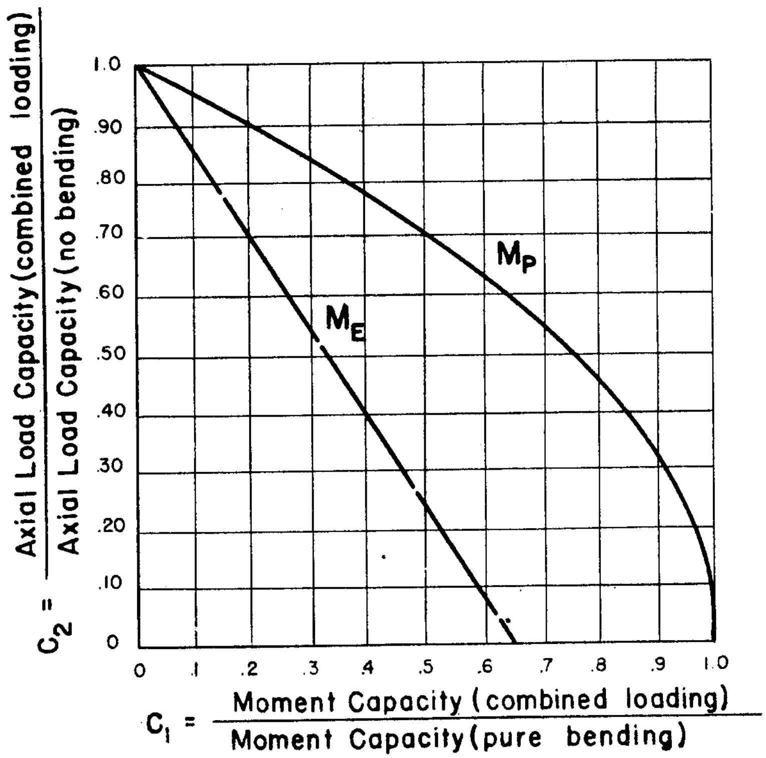
$$M_P = S_y \cdot a(t-a) = S_y \cdot at - S_y \cdot a^2$$

Substituting For a

$$M_P = S_y \cdot t \ \frac{(S_y \cdot t - S_D \cdot t)}{2 S_y} - \frac{S_y (S_D \cdot t - S_y \cdot t)^2}{4 S_y^{\,2}}$$

$$= \frac{t^2}{4 S_y^2}\left(S_y^2 - S_D^2\right)$$

The relationship between the ultimate resisting moment and the axial stress for the assumed rectangular section is shown by curve M_P in figure 2.4.3-8.



COMBINED BENDING AND AXIAL LOAD
HOMOGENEOUS RECTANGULAR MEMBERS

FIG. 2.4.3 - 8

[redacted]

The assumption of full plastic resistance assumes adequate stability against local buckling and strength to resist shear. It appears that for most practical designs standard rolled sections are adequate to develop the full plastic moment. The overall stability of the member is considered later in sections 2.4.5-E and 2.4.5-F.

F. Resistance Function for Steel Members under Combined Bending and Axial Load - Static Load

Curve M_E of figure 2.4.3-8 shows the relationship between the maximum bending-resistance and axial stress with all fibres maintained below the yield point stress.

Similar moment-resistance versus axial-load relationships can be obtained for the rolled shapes generally used in conventional buildings. These members, as shown by figure 2.4.3-3, will develop proportionally smaller increases over the elastic moment resistance M_E than is shown for the rectangular section.

The resistance function for any specific combination of bending and direct stress may be found by plotting points from a curve such as figure 2.4.3-8 versus either time or deflection.

G. Rectangular Reinforced Concrete Sections - Static Load

The composite under-reinforced concrete beam may be expected to behave in a more complicated fashion than the homogeneous steel beams.

The strain in the compression side of the concrete beams increases practically linearly with the distance from the neutral axis. The distribution below the neutral axis, however, is not well defined after the beams are even slightly cracked. As the loading increases tensile cracks appear and the strain distribution below the neutral axis becomes irregular. The position of the neutral axis for low loads may be expected to agree with that calculated using the standard straight-line theory and including tension in the concrete. The neutral axis then rises with increased loading up to a strain of approximately 0.001, at which time the stress-distribution is sensibly linear and the position of the axis agrees quite well with the standard straightline theory excluding tension. Finally, as the strain approaches approximately 0.002 under further loading, the neutral axis for over-reinforced beams begins to fall, dropping considerably before the ultimate failure.

However, as the beams in the test structure are purposely under-reinforced for reasons stated elsewhere, the neutral axis will rise continually with increased load and the load-deformation relationship will be as follows :

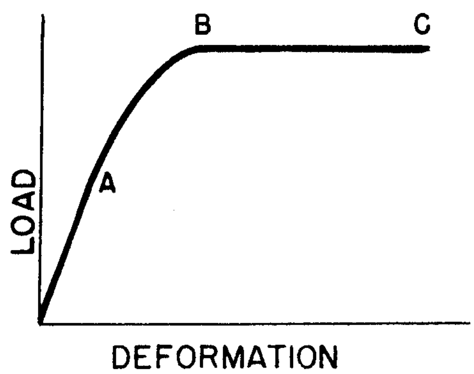


FIG. 2.4.3-9

As the load increases beyond point A the concrete on the tension face will start to crack in the regions of maximum moment, throwing the tensile load to the steel reinforcement. With continuing deformation of the beam the tensile steel will reach its yield point stress at point B, and the steel will continue to elongate, as the beam deflects, with the resistance following some line reasonably close to B—C. The maximum allowable deformation will be reached at point C.

Experimental data shows that the plastic theory developed by Charles S. Whitney (20), (21) gives a much more accurate estimate of the actual ultimate strength of reinforced concrete beams and columns than the standard straight line theory. It provides one simple, consistent method for the entire range of eccentricity of loading from direct axial load to pure bending. Its accuracy was verified by R. H. Evans (22) in 1944 who tested 41 beams with concrete strengths ranging from 975 to 7250 p.s.i. and with steel ratios from 0.0047 to 0.067.

If the beam is under-reinforced then initial failure will occur with the initiation of yield in the tensile steel. At this time the rectangular stress-block decreases in depth until the concrete stress reaches a maximum when the total tensile and compressive forces will be balanced and the lever arm will be maximum. Under further load the concrete will fail, the lever arm will be reduced, and the member will fail.

At failure $$M = bd^2 p f_y \left(1 - \frac{0.5 p f_y}{0.85 f'_c}\right)$$

The limiting ratio of depth for the equivalent rectangular stress block to the effective depth of the beam is 0.537 for the condition of balanced design. Hence $M = 0.333\, bd^2 f'_c$ for compression failures. The critical steel ratio required to develop the full compressive strength of the beam being $P = \frac{0.456 f'_c}{f_y}$

The ultimate moment for the case of beams with compression reinforcement may be estimated by adding the moment of the compression steel, acting at yield point stress, to that of the concrete stress, assuming that the beam is sufficiently reinforced in tension.

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(20) Charles S. Whitney, "Plastic Theory of Reinforced Concrete Design."

(21) Charles S. Whitney, "Application of Plastic Theory to the Design of Modern Reinforced Concrete Structures."

(22) R. H. Evans, "The Plastic Theories for the Ultimate Strength of Reinforced Concrete Beams."

An abbreviated outline of design procedure using the plastic theory is included as follows:

1. Design of Rectangular Members under Bending only:

a. For under-reinforced members without compression steel.

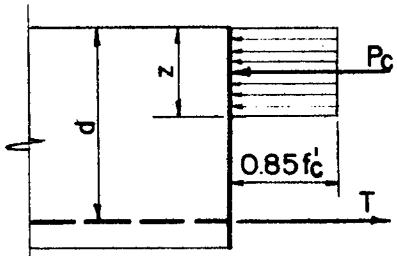


FIG. 2.4.3-10

$T =$ Total tension in tension steel

$P_c =$ Total compression in concrete

$P_c = 0.85 \cdot f'_c \cdot z \cdot b$

where b = beam width

$M_{ult.} = T \cdot (d - \frac{z}{2})$

b. For members with compression reinforcement, the compression steel will act at the yield point.

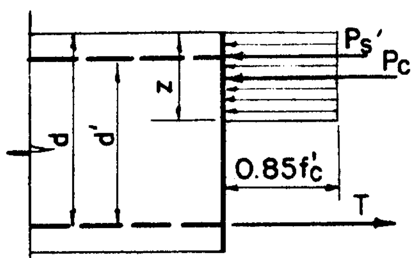


FIG. 2.4.3-11

$M_{ult.} = P_c(d - \frac{z}{2}) + P'_s \cdot d'$

where $P'_s = A'_s \cdot f_y$

2. Rectangular member under bending plus axial load.

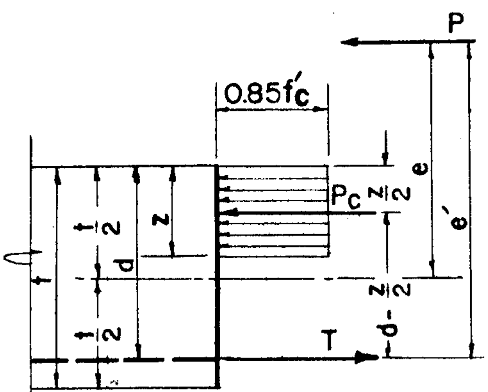


FIG. 2.4.3-12

(a) Compute the moment of the thrust about the tension steel

$M_s = P \cdot e'$

(b) Compute the resisting moment about the tension steel

$M_R = P_c \; (d - \frac{z}{2})$ where

$P_c = 0.85 f'_c \cdot b \cdot z$ and

$Pe' = 0.85 f'_c \cdot b \cdot z(d - \frac{z}{2})$

or directly from 1(a); above

$$z(d - \frac{z}{2}) = \frac{Pe'}{0.85 f'_c b}$$

Where z may be found by trial and error.

(c) Then

$T = P_c - P$ or

$T = 0.85 f'_c \cdot b \cdot z - P$

If compression steel is used the resisting moment of this steel may be deducted before calculating steps (a), (b), and (c).

Where there is sufficient steel to develop a compression failure the maximum value of the resisting moment is:

$$M_c = P \cdot e' = \frac{1}{3} bd^2 f'_c + d' \cdot A' \cdot f_s$$

$$P \cdot e' = P\left(e + d - \frac{t}{2}\right)$$

$$\text{If } d = \left(\frac{t + d'}{2}\right)$$

Then the ultimate load for any eccentricity greater than the eccentricity of the resisting compressive forces may be obtained by,

$$P_{ult.} = \frac{2 A'_s f_s}{\frac{2e}{d'} + 1} + \frac{b t f'_c}{\frac{3te}{d^2} + \frac{6dt - 3t^2}{2d^2}}$$

3. Bending plus axial load at small eccentricities.

The formulas of (1) and (2) give theoretical values when the eccentricity of the load is greater than the eccentricity of the resisting forces in the compression side of the section. For smaller eccentricities the equations must be adjusted by making the allowable thrust $P_{ult.}$ approach the proper value for an axially loaded column as e approaches zero. To do this the final equation of (2) may be adjusted so as to give yield point stress in the steel, as before, while the concrete strength is reduced to $0.85 f'_c$

Then

$$P_{ult.} = \frac{2 A'_s f}{\frac{2e}{d'} + 1} + \frac{b t f'_c}{\frac{3te}{d^2} + 1.178}$$

The above formulae do not allow for increased strength due to the rate of loading.

[redacted]

The relationship between the moment resistance and the axial load for all eccentricities is shown in figure 2.4.3-13.

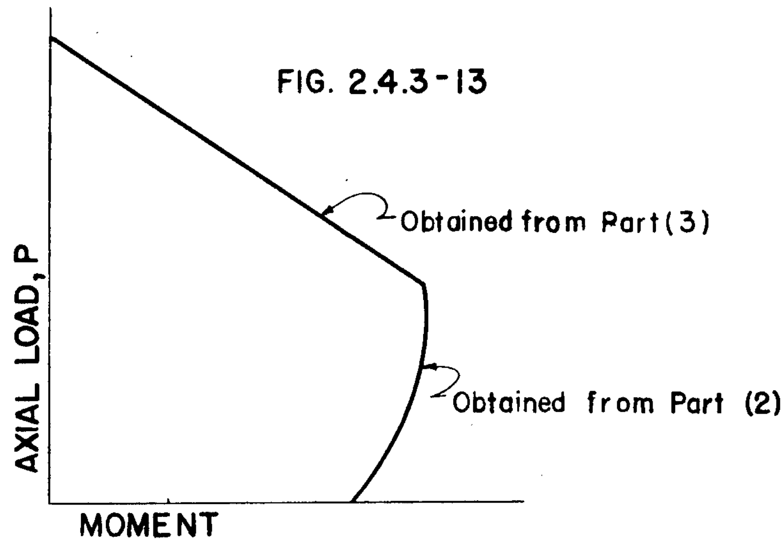


FIG. 2.4.3-13

COMBINED BENDING AND AXIAL LOAD
REINFORCED CONCRETE MEMBERS

Since the total elastic deformation of a member in bending depends on the curvature along its entire length, it is difficult to determine the exact shape of the elastic curvature and the total deflection of the reinforced concrete beams because of the complicated and varying stress relationships at the different sections along the length. However, as the elastic deflection has only a minor effect on the analysis, it is believed that the overall stiffness of the concrete beam within the elastic range may be reasonably approximated by using the moment of inertia of the unreinforced uncracked rectangular section.

H. Bending Resistance Function - Concrete Members Without Axial Load - Dynamic Load

The total deformation of a beam or frame will depend on the mass and strength of the member and on the intensity and duration of the load pulse. As described in Section 2.3.3 under the energy of strain, the velocity and resistance of the beam are zero when the load is applied. The member will begin to accelerate immediately thereafter, the motion being resisted by the inertia forces and the bending moments developed as the member deflects.

The bending resistance will continue to increase finally exceeding the applied load at which time the beam will begin to decelerate. During the deceleration the beam bending resistance will oppose both the applied load and the inertia forces.

In order to evaluate the effect of dynamic loads on the shape and magnitude of the resistance-function and to investigate the probable equivalent mass factors, a number of studies were made based on a series of tests and reports (7) obtained from the Massachusetts Institute of Technology.

These Massachusetts Institute of Technology tests were made on beams of different size, length and percentages of reinforcement. The given data included time-pressure and time-deflection curves as shown in figure 2.4.3-14.

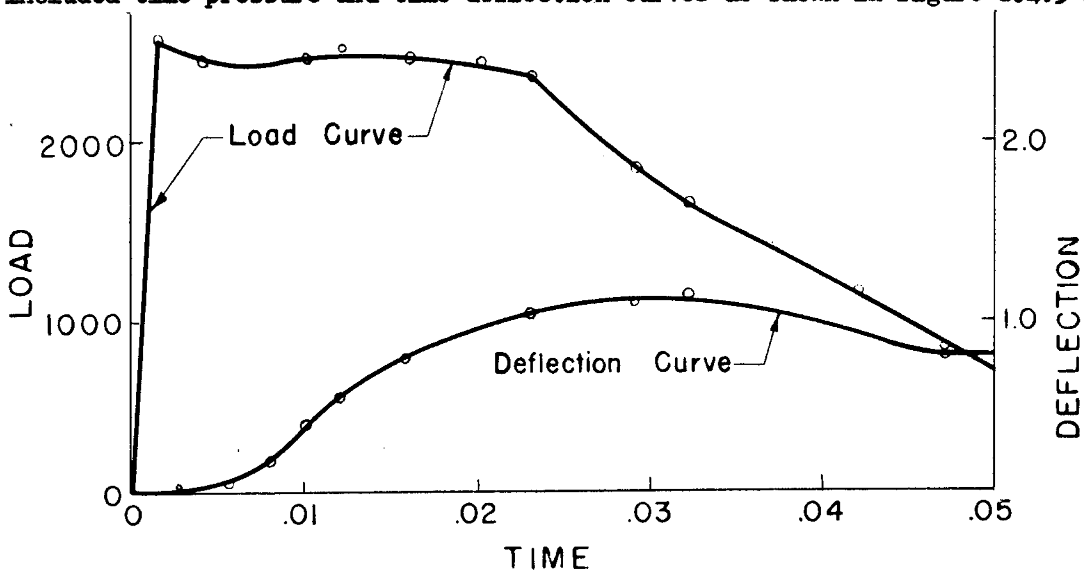


FIG. 2.4.3-14

From these load-time and deflection-time curves the resistance of each time increment was determined as follows:

$$F = m_e a \qquad\qquad F = P - R$$

$$P - R = m_e \frac{\Delta v}{\Delta t}$$

$$R = P - m_e \frac{\Delta v}{\Delta t}$$

F = Statically unbalanced load
m_e = Equivalent mass
a = Acceleration
P = Total applied load
R = Resistance of beam
Δv = Change in velocity during time interval, Δt
Δt = Time interval
m = Total mass of the beam

(7) J. B. Wilbur, R.J. Hansen, K. Steyn, "Behavior of Reinforced Concrete Structural Elements Under Long Duration Impulsive Loads."

[redacted]

The variation of **R** with time is derived by the method outlined in the table of figure 2.4.3-15, in which the double differentiation required to compute $\frac{\Delta V}{\Delta t}$ is performed by a numerical step-by-step procedure.

The equation $a = F/m_e$ expresses the linear acceleration of a free body of mass, m_e with the applied force acting through the centroid of the mass. Though the beam is not a free mass and the velocity and acceleration varies along its length, the acceleration and deflection at the centerline may be obtained by using a particular fraction m_e of the total mass m in the solution of the above equation. This fraction will be referred to as the equivalent mass factor. To obtain column 8 in figure 2.4.3-15 an equivalent mass factor of 1/3 was used. In later sections a detailed discussion of this mass factor will be presented.

1	2	3	4	5	6	7	8	9	10
t	Δt	x	Δx	$\bar{V}$	V_n	ΔV	$P-R$	P	R
t_o		0			0				
	—		—	—		—	—	—	—
t_1		—			—				
t_{f-1}		—			—				
	—		—	—		—	—	—	—
$t_f{}^1$		—			—				
Time	Time Interval $= t_n - t_{n-1}$	Deflection Given By Deformation – Time Curve	Change in Deflection $= X_n - X_{n-1}$	Average Velocity During Time Interval $= \frac{\Delta X}{\Delta t}$	Velocity At Beginning & End Of Time Interval	Change In Velocity During Time Interval $= V_n - V_{n-1}$	Net Translational Force $= m \cdot \frac{\Delta V}{\Delta t}$	Given in Pressure – Time Curve	$P - (P - R)$

1. Analysis Complete At t_f

FIG. 2.4.3-15

[redacted]

The individual values of **R** obtained from the test data by use of the table in figure 2.4.3-15 were quite erratic for the separate time increments. To obtain a smooth resistance curve, the resisting or **R** forces were integrated with respect to time by summing the individual values of $R \Delta t$ giving the resisting impulse shown in figure 2.4.3-16. By drawing the straight lines **OA** and **AB** through the points and then differentiating with respect to time, a final simple and usable resistance curve was obtained in

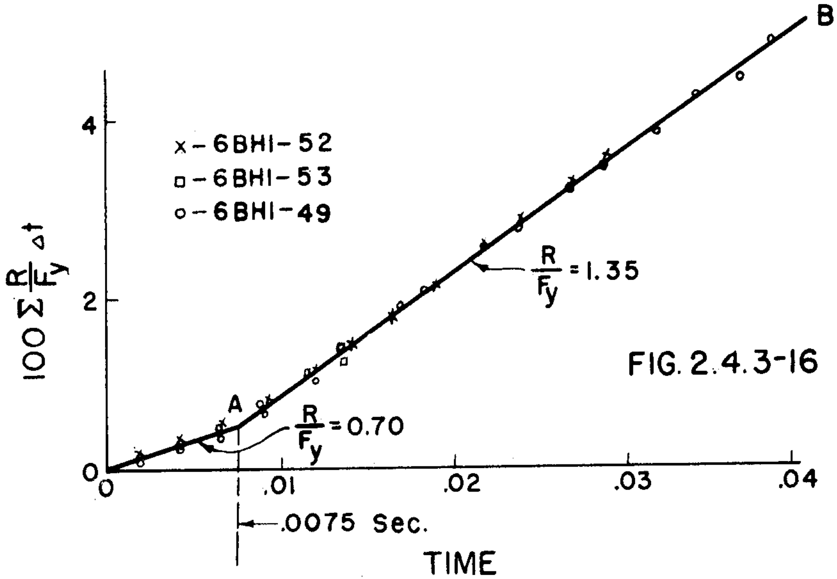


FIG. 2.4.3-16

figure 2.4.3-17. As shown, this curve shows a 35% increase in ultimate strength beyond the static strength and also indicates a variable resistance best represented by another straight line from the time of the initiation of the loading to the point of maximum resistance.

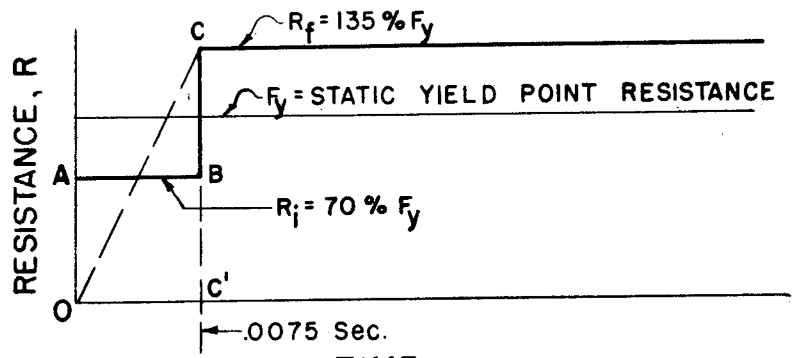


FIG. 2.4.3.-17

To check the reliability of this average resistance-time function, time deflection curves were computed for several of the Massachusetts Institute of Technology test beams using the above resistance function, the actual loadings of the test, and reversing the sequence of the operations shown in the table of figure 2.4.3-15.

[redacted]

The given pressure, calculated deflection, and measured deflection for a typical test beam is shown in figure 2.4.3-18.

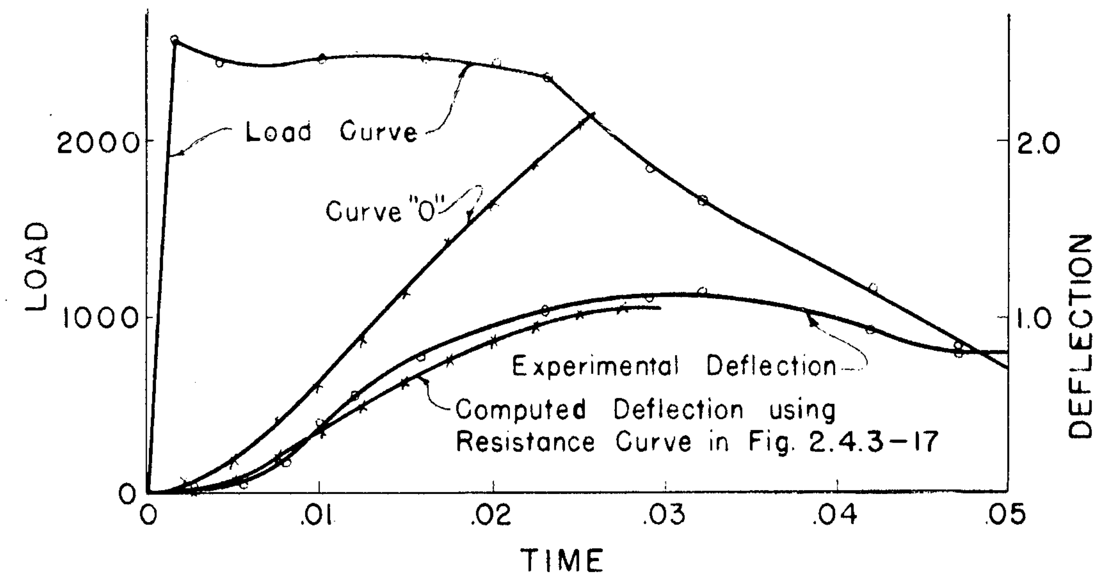


FIG. 2.4.3-18

As shown, the computed time-deformations obtained using the average resisting function are in close agreement with the measured deflections of the test.

In order to compare the derived dynamic resistance-time curve with a resistance function similar to the static resistance curves which assume a linear stress-strain relationship from zero stress to the yield point, the deflection-time curves for the same test beam were recomputed using the resistance curve of figure 2.4.3-19. These results are indicated by curve **O** in figure 2.4.3-18. It should be noted that, though the area of the elastic resistance, $OA\delta_y$ of figure 2.4.3-19, is approximately equal to the area **OABC** of figure 2.4.3-17, the latter distribution of resistance gives superior results for estimating the total deflection.

I. <u>Summary - Bending Resistance Function - Concrete **Members** Under Dynamic Loads</u>

It was initially expected that the resistance would increase linearly with strain at stresses below the yield point, but careful analysis of the laboratory tests indicated a more nearly constant resistance during this time.

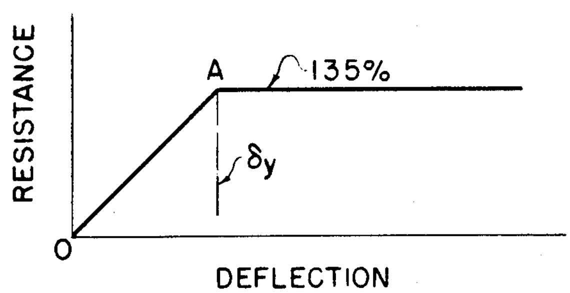
More information and data would be useful on this subject, for this method though seemingly justified for certain particular members is lacking in generality until more abundant proof is available.

[redacted]

The resistance curve adopted and shown in figure 2.4.3-17 shows the yield point as a function of time. It was at first expected that this time of yield could not be arbitrarily extended to wall and beam elements of different size, length, and weights. To find the probable variation in time necessary to reach plastic yield for the different parts of the test building, several wall and roof members were investigated using a resisting curve of the type shown in figure 2.4.3-17. These studies indicated that the error in yield point time would not appreciably change the effective resistance function.

The rapid loading tests of California Institute of Technology and Massachusetts Institute of Technology both indicate that under extremely high rates of loading the linear stress-strain relationship may be disturbed and the stress-time resistance curve is controlled by the time required to initiate plastic strain.

In contrast to the analysis for the quick acting panels and slabs, the deflections of the heavier slow-moving frames were computed using the conventional type stress-strain resistance function of figure 2.4.3-19 rather than the above described resistance-time curves. Unless the pressure-time curves and member characteristics change appreciably from those of the test structure, intermediate cases are of academic rather than practical interest.



RESISTANCE - DEFLECTION CURVE

FIG. 2.4.3-19

If the members are being investigated for a blast pressure of an intensity which will cause only small plastic strains or for blast pressures which have a time of rise, then the rate of loading will be slower and the use of a linear stress-strain curve and the tabular method of 2.5.3-C is recommended. It may be necessary in these cases to consider the variation in the resistance function due to the difference in the time at which the different parts of the member reach the yield point stresses. This condition, while not important in estimating the total blast capacity, is of interest in the investigation of the behavior of members at loads different than the critical intensity.

[redacted]

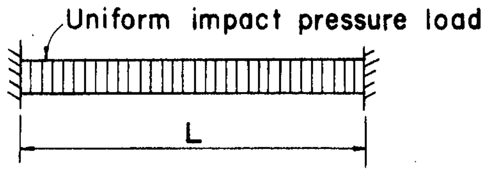


FIG. 2.4.3-20

For the beam in figure 2.4.3-20, which is assumed to have equal strength against positive and negative moments, the first yield points, M_{YP}, are developed at the fixed supports at time t_1 (see figure 2.4.3-21). The maximum deflection at this time is x_1 and the corresponding resistance is W_{R_1}. Up to time, t_1, the equivalent mass is that of an elastic fixed-end beam.

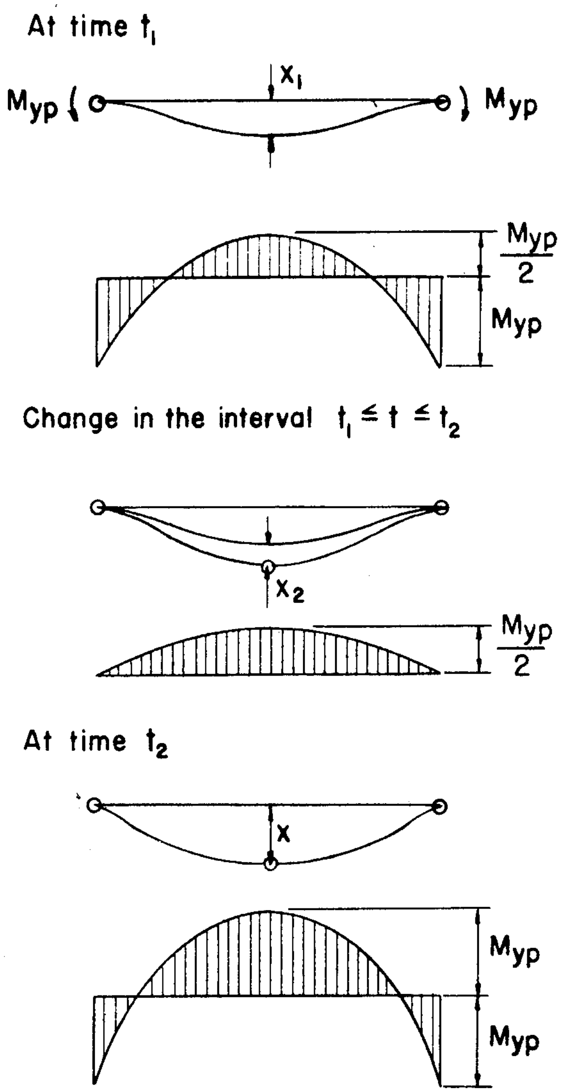
After, t_1, the beam is assumed to be simply supported with a constant moment applied at the ends. At some time, t_2, a plastic moment will also develop at the midspan and the corresponding deflection and resistance is X and W_R respectively. During the interval, $t_2 - t_1$, the equivalent mass will be that of a simple beam and the change in resistance and deflection is W_{R_2} and X_2 respectively.

All the data required to determine the resistance function prior to development of the full plastic resistance is shown in figure 2.4.3-21. If the resistance function is plotted to scale as shown in figure 2.4.3-22, line OABC, the work involved in a step-by-step analysis is facilitated.

In the design of reinforced concrete members the beam will probably be proportioned so that positive and negative plastic hinges develop simultaneously, in which case, the resistance function would have only one change in direction as shown by line OAD. This simultaneous yielding at all critical points is advantageous in limiting the cracking caused by one set of moments having plastic hinges developed before full beam resistance is achieved.

The resistance function for the steel members is assumed to increase linearly from zero stress to the dynamic yield point value, the dynamic yield point being obtained from figure 2.4.2-5 which shows the recommended increase in yield point properties under rapid rates of loading.

While it is believed that the resistance function for steel members will also be of a modified form, somewhat similar to that of the concrete members, the information necessary to establish these characteristics is not available at the present time.



$$W_{R_1} = \frac{12 M_{yp}}{L}$$

$$x_1 = \frac{W_{R_1} L^3}{384 EI}$$

$$W_{R_2} = \frac{4 M_{yp}}{L}$$

$$x_2 = \frac{5 W_{R_2} L^3}{384 EI}$$

$$W = W_{R_1} + W_{R_2} = \frac{16 M_{yp}}{L}$$

$$x = x_1 + x_2 = \frac{L^3 (W_{R_1} + 5 W_{R_2})}{384 EI}$$

FIG. 2.4.3-21

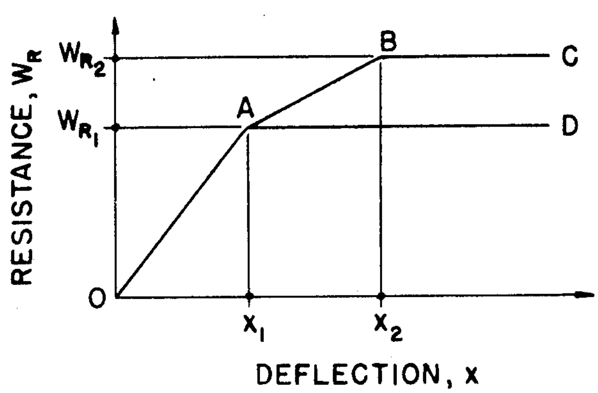


FIG. 2.4.3-22

J. Summary - Bending Resistance Function - Steel Members Under Dynamic Loads

The resistance function for the steel members is assumed to increase linearly from zero stress to the dynamic yield point value, the dynamic yield point being obtained from figure 2.4.2-5 which shows the recommended increase in yield point properties under rapid rates of loading.

While it is believed that the resistance function for steel members will also be of a modified form, somewhat similar to the concrete members, the information necessary to establish these characteristics is not available at the present time.

2.4.4 Equivalent Mass Factors

Before proceeding with the analysis of typical beam, frame, and slab members, it would seem advantageous to discuss the equivalent mass factors used in the analysis of the buildings. The object of this investigation being, as in all other recommended design phases, the derivation of a simple, usable value which will permit a fairly accurate estimate of blast-pressure resistance. To this end a number of studies were conducted to determine the most acceptable value for this effective mass.

From purely theoretical considerations the value of the equivalent mass will vary with the beam curvature and displacement. Ignoring wave propagation and dynamic vibrations of several possible modes, a simply supported beam may be considered to have an elastic curvature as shown in figure 2.4.4-1.

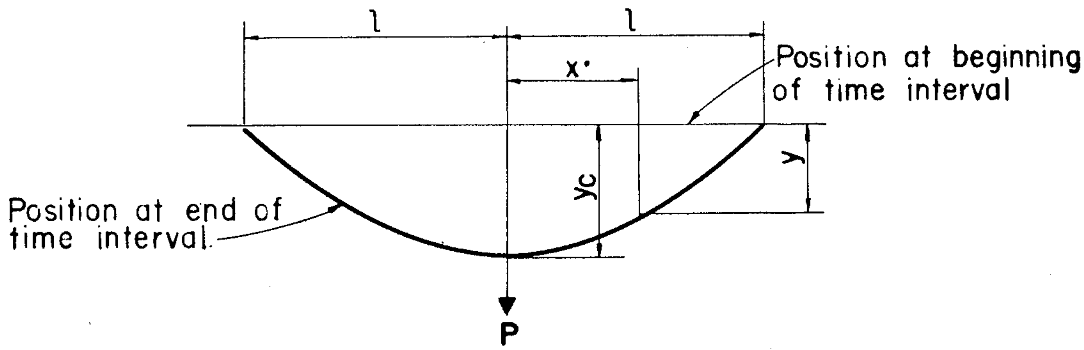


FIG. 2.4.4-1

The inertial resistance of an element of mass (mdx) at point x is $F=(mdx)\ddot{y}$.
The total dynamic resistance per half is:

$$R_d = \int_0^l m\ddot{y}\,dx$$

F = Net applied force
m = Mass per unit length
y = Deflection of the beam at x
$\ddot{y}$ = Acceleration of an element of the beam at point x

[redacted]

Assuming that the curvature will remain the same after the first infinitesimal deflection and that the deflection at any time may be expressed by

$$y = y_C \left[1 - \left(\frac{x}{l}\right)^n\right]$$

This expression becomes useful in that measurements at one other point besides the centerline will establish the effective mass factor. If y is measured at $\frac{x}{l} = \frac{1}{2}$

then $$n = 3.32 \log\left(\frac{1}{1 - y/y_C}\right)$$

The ratio of kinetic energy imparted to an element at x compared to the kinetic energy imparted to the same mass at the centerline would be:

$$\left(\frac{v_x}{v_c}\right)^2 = \left(\frac{\dot{y}_x}{\dot{y}_c}\right)^2 = \left(\frac{y_x}{y_c}\right)^2$$

The ratio K of the average energy imparted along the total length of the beam to the energy imparted to a unit mass at midspan is

$$K = \frac{1}{2l}\int_{-\ell}^{+\ell}\left(\frac{y_x}{y_c}\right)^2 dx = 1 - \frac{2}{n+1} + \frac{1}{2n+1}$$

Values of K are listed for various values of n in the following table of figure 2.4.4-2

$\frac{y \,@\, \frac{l}{2}}{y_c}$	.50	.52	.54	.56	.58	.60	.75	.90
n	1.00	1.06	1.12	1.18	1.25	1.32	2.00	3.32
K	.33	.34	.37	.38	.40	.41	.53	.67

FIG. 2.4.4—2

[redacted]

Values of n = 1.00 and n = 2.00 correspond to triangular and second degree parabola beam profiles respectively.

At lower values of stress, when the load is first applied, the curvature may be calculated from the applied moment, assuming a linear stress distribution and assuming that the values of E and I can be predicted with sufficient accuracy. During this time the elastic curvature is approximately parabolic in shape and the effective mass may be expected to be approximately 1/2 of the total mass n = 2.00 After yield point stress is reached at midspan, however, the moments along the beam will remain constant, consistent with the adopted resisting function shown in figure 2.4.3-19. The values of M/EI and hence the curvature along the beam will not change during the succeeding time intervals and n will be equal to 1.00.

To substantiate the above conclusions an analysis was made of the Massachusetts Institute of Technology report, "A Summary of an Investigation of the Shape of the Deflection Curve as Affecting the Apparent Mass of a Freely Supported Beam".(23) In this report data from a number of beam tests was presented wherein the centerline and quarterpoint deflections were recorded with respect to time for beams deflecting under dynamic loads. As the beam characteristics were not given, it was not possible to establish the degree of plastic deformation obtained in the tests. However, examination of the data indicates that the steel strains may not have been carried very far into the plastic range. In the Massachusetts Institute of Technology report effective mass versus centerline deflection and effective mass versus time were calculated and plotted as shown in figures 2.4.4-3 and 2.4.4-4 respectively.

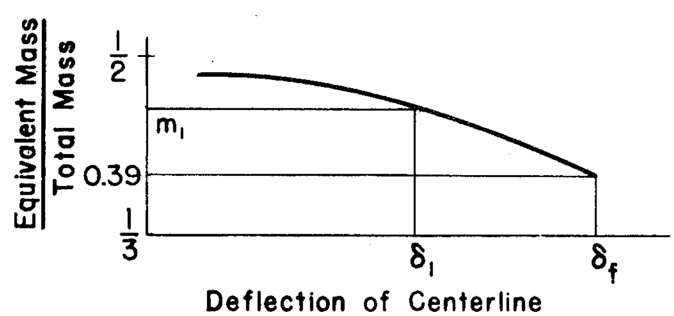


FIG. 2.4.4 -3

(23) Staff, Massachusetts Institute of Technology, Department of Civil and Sanitary Engineering, "A Summary of an Investigation of the Shape of the Deflection Curve as Affecting the Apparent Mass of a Freely Supported Beam".

[redacted]

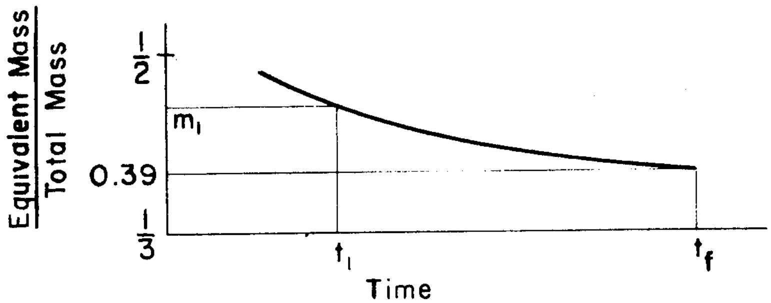


FIG. 2.4.4-4

The particular value of the effective mass m shown in figures 2.4.4-3 & 4 for a specific value of the deflection at the centerline δ or any particular time t_1 is the average effective mass factor from the position at rest before the load was applied to the deflection δ or time t_1 plotted. In other words the effective mass averaged from the beginning to the end of the test was approximately 0.39 of the total mass.

To find the instantaneous effective mass for each small increment of time during the deflection of the beam, additional studies were made using the same test data. In this study the change in curvature between successive increments of time was computed with results as plotted in figure 2.4.4-5.

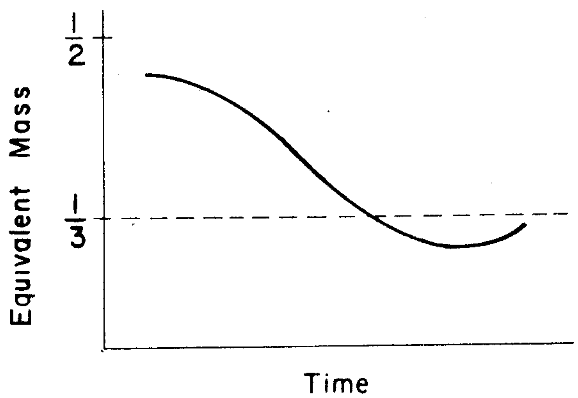


FIG. 2.4.4-5

This study shows that the apparent mass factor corresponds closely to the expected 1/3 for strains beyond the elastic range.

To find the effect of different mass factors, the same pressure loading and two extreme mass factors using $n = 1.00$ and $n = 2.00$ corresponding to continuous plastic and continuous elastic behavior respectively, were used in a step-by-step derivation of the resistance function. The two resistance

[redacted]

functions thus obtained are shown in figure 2.4.4-6.

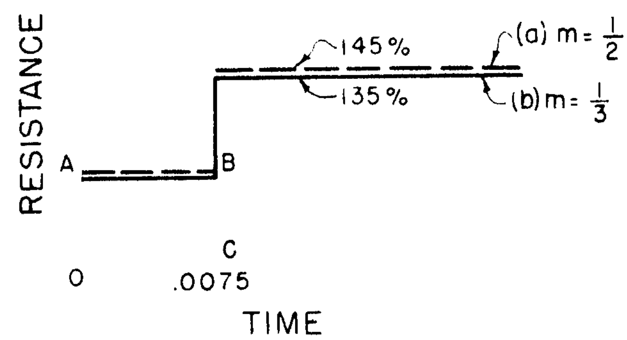


FIG. 2.4.4-6

Either resistance curve used with its appropriate mass factor will give the deformations for the particular test load shown in figure 2.4.3-18 from which both were derived.

The difference between the resisting functions was slight.

The study was then continued by varying the equivalent mass factors in the elastic range as shown by lines a, b and c in figure 2.4.4-7. The integral of the corresponding resistance curve with respect to time for each case is shown in figure 2.4.4-8.

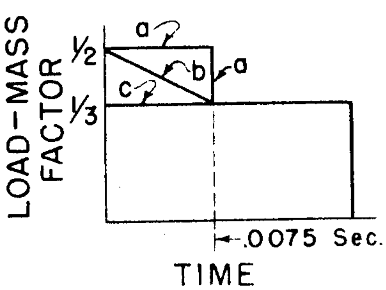


FIG. 2.4.4-7

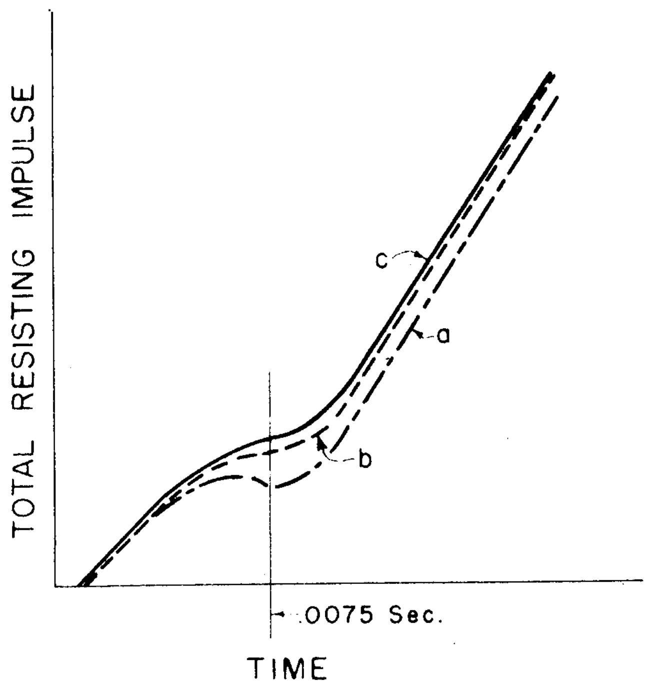


FIG. 2.4.4-8

[redacted]

Figures 2.4.4-7 and 2.4.4-8 show that variation of the effective mass factor for simply supported beams within the extreme upper and lower limits seems to have only minor effects on the required design strength. It is therefore recommended that an effective mass factor of 1/3 be used throughout the range of deflections because of the desirable simplification of the design analysis thus attained.

Continuous Beams: Continuous beams are subject to a reversed curvature within the elastic range as shown in figure 2.4.4-9.

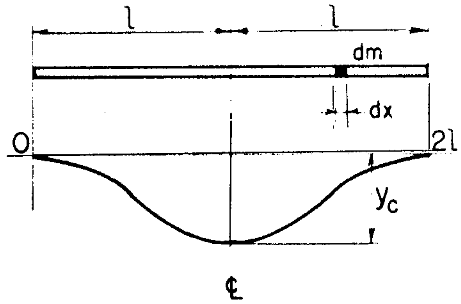


FIG. 2.4.4-9

$$y = \frac{y_c}{2}\left(1 - \cos.\frac{\pi x}{l}\right)$$

$$v = \frac{dy}{dt} = \frac{v_c}{2}\left(1 - \cos.\frac{\pi x}{l}\right)$$

$$KE = \frac{v_c^2}{4}\int_0^{2l} \frac{m\,dx}{2}\left(1 - \cos.\frac{\pi x}{l}\right)^2 = 0.375\, m\, l\, v_c^2$$

Thus within the elastic range the initial equivalent mass factor may be expected to be approximately 0.375, or slightly less than the value of 0.500 for the elastic range of the simple beam. As the deflection continues the yield point will be reached at the supports and further changes in the beam may be expected to be similar to those for the simple beam described previously. When the yield point at the center line is also reached the end and centerline moments will not change and the effective mass factor will again be 1/3. Where the greater part of the deflection is within the plastic range, the use of 1/3 throughout the deflection is again recommended as being even closer an approximation than for the case of the simple beam.

[redacted]

2.4.5 Detailed Methods of Analysis

A. Members Resisting Loads in Bending

General Method of Derivation

In the previous derivations, use of the equation $F\,dt = m_e\,dv$ in a step-by-step computation for beam deflection, has assumed that an unbalanced force would impart a linear momentum to an equivalent mass of the beam shown in figure 2.4.5-1 below. In the following section a method of computing equivalent load mass ratios for use in step-by-step procedures will be presented assuming only that plastic deformation occurs throughout the time of motions as discussed previously.

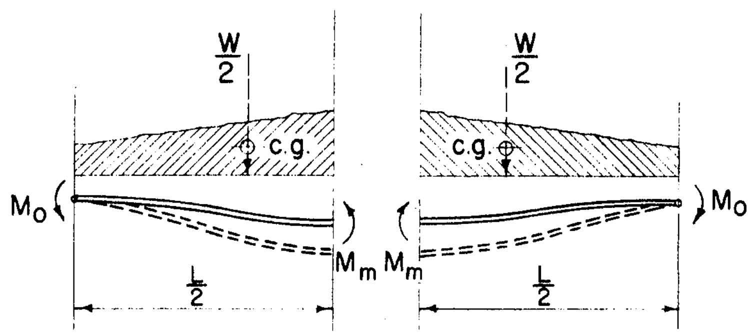


FIG. 2.4.5-1

If we assume that plastic hinges have developed at the point of maximum moment, the two halves of the member may be considered to be rotating about the support as rigid bodies under the action of the unbalanced dynamic load.

The acceleration of a member or, in the case of a beam or slab, the acceleration of the midspan of the member, will depend on the equivalent mass, the loading, and the ultimate resistance. For convenience, a single term called the <u>load-mass factor</u> will be derived which will include the mass effect, the loading effect, and the effect of the type of member being considered. Having the load-mass factor, the total load, and the total resistance, each type of a member may be solved by a single process and tabular arrangement.

As the beam is not free to deflect at the supports, a dynamic reaction, in addition to the bending reaction will be developed unless the force vector passes through the center of percussion of the element. If the beam is considered as a free member cut at midspan, with each half rotating about its support as shown in the figure 2.4.5-4 below, the total reaction, S , at the supports, will be equal to the shear, R , required to develop the bending moment, M_C , at the midspan, plus the force, F_2 , necessary to hold the end of the accelerating beam at point O . The latter part of the reaction will depend, at any instant, on the net force causing angular acceleration and may be found as follows:

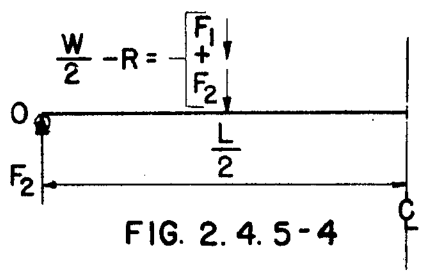


FIG. 2.4.5-4

F_1 = Unbalanced force on each half of the beam causing linear acceleration of each half of the beam.

F_2 = Dynamic reaction for each half of the beam required to maintain the supported end of point O during rotation.

I_C = Moment of Inertia of each half of the beam about its center of gravity.

Several conditions of loading and end restraint for various types of members will be considered as follows:

1. <u>Simple beam or slab with a concentrated load at midspan</u>

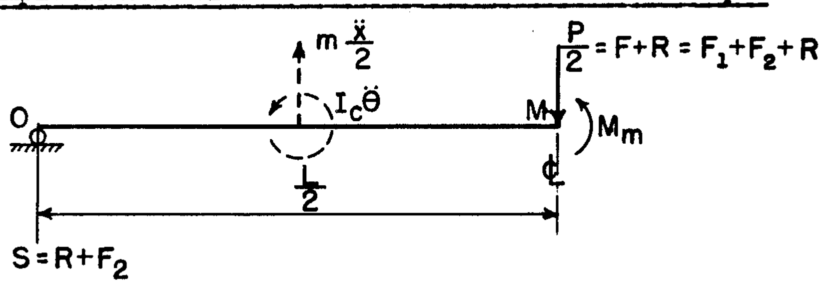


FIG.2.4.5-2

S = Total reaction at the support.

R = End shear corresponding to static resistance.

P = Concentrated load at midspan.

P/2 = Load to each side.

[redacted]

M_m = Moment at midspan. $= \frac{L}{2} \cdot R$,

F = Unbalanced dynamic load to each side. $= \frac{P}{2} - R$

F_1 = Unbalanced force on each half of the beam causing linear acceleration of each half of the beam.

F_2 = Dynamic reaction for each half of the beam required to maintain the supported end of point O during rotation.

I_c = Moment of inertia about the center of gravity of one half the beam.

θ = Angular acceleration of half the beam.

$\ddot{x}$ = Linear acceleration of the beam at midspan.

m = Mass of $\frac{1}{2}$ of the beam.

m_e = Equivalent mass of $\frac{1}{2}$ of the beam.

L = Length of span.

$$\Sigma \text{ Moments at } O = 0 = \frac{P}{2} \cdot \frac{L}{2} - M_m - m\frac{\ddot{x}}{2} \cdot \frac{L}{4} - I_c \ddot{\theta}$$

$$M_m = \frac{L}{2} \cdot R$$

$$I_c = \frac{1}{48} m L^2$$

$$\theta = \frac{2\ddot{x}}{L}$$

therefore $\left(\frac{P}{2} - R\right) \frac{L}{2} = \frac{mL}{8} \cdot \ddot{x} + \frac{mL^2}{48} \cdot \frac{2\ddot{x}}{L} = \frac{mL}{6} \ddot{x}$

and $F = \frac{1}{3} m\ddot{x} = m_e \ddot{x}$ Where $\underline{m_e = \frac{1}{3} m}$ and the Load-Mass Factor $= \frac{1}{3}$

$$\Sigma \text{ Moments at } M = 0 = (R + F_2)\frac{L}{2} + m\frac{\ddot{x}}{2} \cdot \frac{L}{4} - I_c\ddot{\theta} - M_m$$

$$F_2 \frac{L}{2} + \frac{mL}{8}\ddot{x} - \frac{mL}{24}\ddot{x} = \frac{L}{2}\left(F_2 + \frac{1}{6} m\ddot{x}\right) = 0$$

therefore $\underline{F_2} = -\frac{1}{6} m \ddot{x} = -\frac{1}{2} m_e \ddot{x} = \underline{-\frac{1}{2} F}$

$$\underline{S = R - \frac{1}{2} F}$$

$$\Sigma \text{ Vertical Forces} = 0 = R + F_2 \cdot m\frac{\ddot{y}}{2} - F_1 - F_2 - R$$

therefore $\underline{F_1} = \frac{1}{2} m \ddot{x} = \frac{3}{2} m_e \ddot{x} = \underline{\frac{3}{2} F}$

[redacted]

2. Simple beam with a uniform load

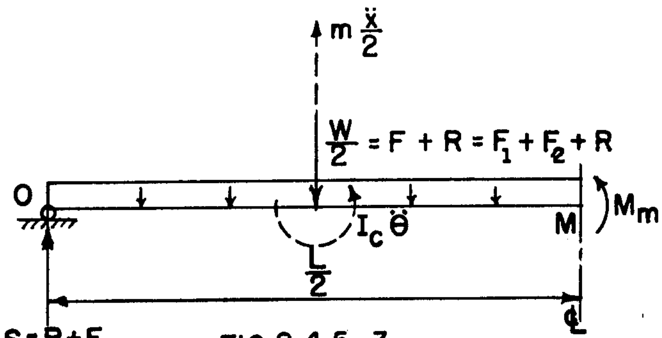


FIG. 2.4.5-3

W = Total load on the beam

M_m = Moment at midspan $= \frac{L}{4} \cdot R$

F = Unbalanced dynamic load to each side $= \frac{W}{2} - R$

At any instant after the center deflection exceeds the yield point deformation:

$$\Sigma \text{ Moments at } O = 0 = \left(\frac{W}{2} - R\right)\frac{L}{4} - m\frac{\ddot{x}}{2} \cdot \frac{L}{4} - I_c\ddot{\theta}$$

$$\left(\frac{W}{2} - R\right)\frac{L}{4} = \frac{mL}{6}\ddot{x}$$

and

$$F = \frac{2}{3}m\ddot{x} = m_e\ddot{x} \qquad \text{where } \underline{m_e = \frac{2}{3}m}$$

and the Load-Mass Factor for simple beams subjected to a uniform load equals $\frac{2}{3}$

$$\Sigma \text{ Vertical Forces } = 0 = R + F_2 + m\frac{\ddot{x}}{2} - F_1 - F_2 - R.$$

therefore

$$\underline{F_1} = \frac{1}{2}m\ddot{x} = \frac{3}{4}m_e\ddot{x} = \underline{\frac{3}{4}F}$$

$$\Sigma \text{ Moments at } M = 0 = (R + F_2)\frac{L}{2} + m\frac{\ddot{x}}{2} \cdot \frac{L}{4} - I_c\ddot{\theta}$$

$$- (F_1 + F_2 + R)\frac{L}{4} - M_m$$

$$F_2\frac{L}{4} + \frac{mL}{8}\ddot{x} - \frac{mL}{24}\ddot{x} - F_1\frac{L}{4} = \frac{L}{4}\left(F_2 + \frac{1}{3}m\ddot{x} - F_1\right) = 0$$

therefore

$$\underline{F_2} = F_1 - \frac{1}{3}m\ddot{x} = \frac{3}{4}m_e\ddot{x} - \frac{1}{2}m_e\ddot{x} = \frac{1}{4}m_e\ddot{x} = \underline{\frac{1}{4}F}$$

$$\underline{S = R + \frac{1}{4}F}$$

[redacted]

3. Dynamic End Reaction

This paragraph has been omitted, since treatment of dynamic end reaction has been included in paragraphs 1 and 2, above.

(See discussion
on Page 66 and
Figure 2.4.5-4)

4. Fixed end beam or slab with either a concentrated load at midspan or a uniform load

A fixed end beam or slab will act in a similar manner to the simple beam except that the moment at the support M_0 will not be zero. As the load is applied both M_0 and M_c will increase within the elastic range. As the load increases to the point where $\frac{\text{Load x Span}}{12}$ for a uniform load and $\frac{\text{Load x Span}}{8}$ for a concentrated load at midspan, exceeds the yield point moments at the supports, the ends of the member will enter the plastic range and will start to act as plastic hinges. The moment at the center may also reach yield point at this time - for concentrated loads or if the center is designed for a smaller moment capacity - however if the member is of uniform strength the moment at the center will continue to increase under further strain until it too becomes plastic. At this time, with both M_0 and M_c plastic, the unbalanced force causing angular acceleration will be $P/2 - R$ or $W/2 - R$ where $R = (M_0 + M_c)/\frac{L}{2}$ for a concentrated load and $R = \frac{(M_0 + M_c)}{L/4}$ for a uniform load at midspan.

The load-mass factors will be the same as for the simple beam, i.e. 1/3 for the concentrated load and 2/3 for the uniform load. The only change will be the value of the resistance force described above.

5. Two-way square slabs

The ultimate two-way panels seemed to offer so many difficulties for elastic analysis under dynamic loads that it was decided to approach the panel analysis in the same manner as used for the beams and to check these results against standard engineering practice. For the analysis it is assumed that the lines of failure may be predicted and that an empirical time-resistance function may be used throughout the slab deformation.

[redacted]

Considering a square panel, as shown in figure 2.4.5-5a:

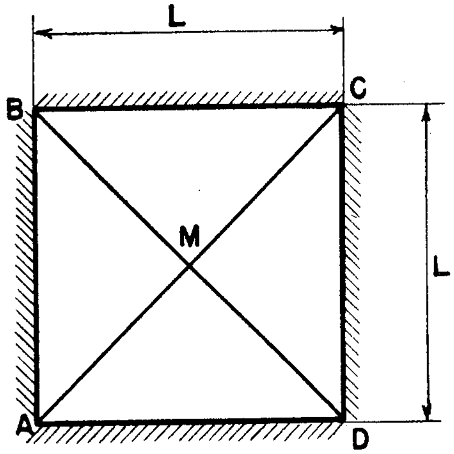


FIG. 2.4.5-5a

Assume that yield points will be reached in the top steel of the slab along lines AB, BC, DC, and DA and at the bottom steel along diagonals AMC and BMD. Assume a uniform load W distributed over the entire slab.

Consider one of the quarters ABM of the slab as shown in figure 2.4.5-5b:

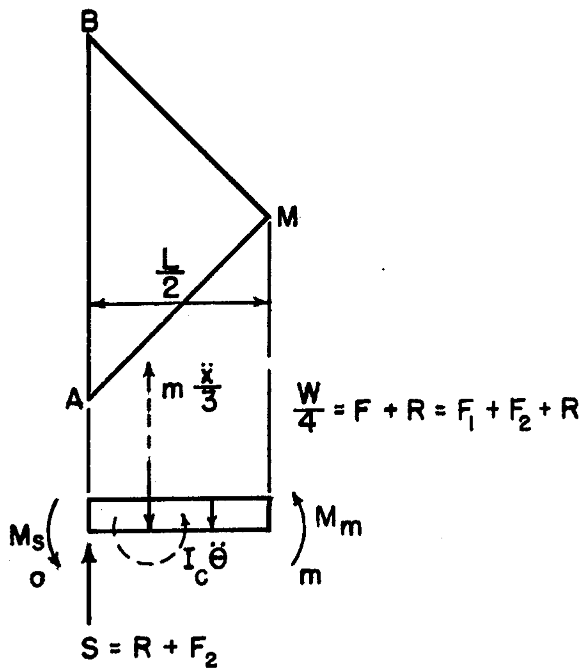


FIG. 2.4.5-5b

As each quarter rotates about its particular support under the influence of the load W, the shears along lines BM and AM will be near zero because of symmetry of deflection in the four identical and equally loaded quarters.

[redacted]

At any instant, the quarter panel will rotate about the line AB as follows:

M_s = Total moment along failure plane of support of triangle AMB

M_m = Component, perpendicular to the supporting edge, of all the moments along the interior failure planes of the triangle

m = Mass of $\frac{1}{4}$ of panel ABCD

R = Static resistance of the triangle AMB $= \frac{M_s + M_m}{L/6}$

W = Total load on panel

F = Unbalanced dynamic load to each quarter panel $= \frac{W}{4} - R$

F_1 = Unbalanced force causing linear acceleration of each quarter panel

F_2 = Dynamic reaction required to maintain the supported side at O during rotation

L = Length of side

$\ddot{\theta}$ = Angular acceleration of triangular segment

$\ddot{x}$ = Linear acceleration of midpoint of panel

I_c = Moment of inertia about an axis parallel to the support and through the center of gravity of the quarter panel

then:

$$\Sigma \text{ Moments at } O = 0 = \frac{W}{4} \cdot \frac{L}{6} - (M_s + M_m) - m\frac{\ddot{x}}{3} \cdot \frac{L}{6} - I_c\ddot{\theta}$$

$$\left(\frac{W}{4} - R\right)\frac{L}{6} = \frac{mL}{18}\ddot{x} + \frac{mL^2}{72} \cdot \frac{2\ddot{x}}{L} = \frac{mL}{12}\ddot{x}$$

or

$$F = \frac{1}{2} m\ddot{x} = m_e\ddot{x} \qquad \text{Where } m_e = \frac{1}{2} m$$

Therefore the Load-Mass Factor for a square two-way panel is $\frac{1}{2}$.

A step-by-step analysis may now proceed on the same basis as that shown in the table of figure 2.4.5-9 for a beam or one-way slab.

The supports along the slab edges must be capable of sustaining a load equal to the static moment resistance of the adjacent segment plus any additional dynamic reaction resulting from the rotation of the member. If we assume negligible settlement of the supports and use the same notation as was used for the rectangular shaped member, the derivation for the dynamic reaction of a triangular segment is as follows:

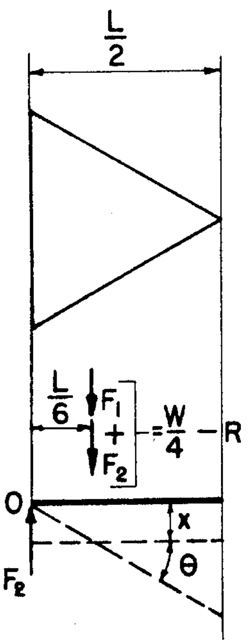


FIG. 2.4.5-5

Σ Vertical Forces

$$= 0 = R + F_2 + m\frac{\ddot{x}}{3} - F_1 - F_2 - R$$

so

$$\underline{F_1} = \frac{1}{3} m\ddot{x} = \frac{2}{3} m_e\ddot{x} = \underline{\frac{2}{3} F}$$

Σ Moments at $M = 0$: $(R + F_2)\frac{L}{2}$

$$+ m\frac{\ddot{x}}{3} \cdot \frac{L}{3} - I_c\ddot{\theta} - (F_1 + F_2 + R)\frac{L}{3}$$

$$- (M_s + M_m)$$

then

$$F_2\frac{L}{6} + \frac{mL}{9}\ddot{x} - \frac{mL}{36}\ddot{x} - F_1\frac{L}{3} =$$

$$\frac{L}{6}\left(F_2 + \frac{1}{2} m\ddot{x} - 2F_1\right) = 0$$

Therefore $\underline{F_2} = 2F_1 - \frac{1}{2} m\ddot{x} = \frac{4}{3} m_e\ddot{x} - m_e\ddot{x} = \frac{1}{3} m_e\ddot{x} = \underline{\frac{1}{3} F}$

or

$$\underline{S = R + \frac{1}{3} F}$$

Assuming that the above procedure will succeed in allowing for all dynamic effects, there still remains the necessity for predicting the instantaneous distribution of resisting moment along the edges and diagonals. Assuming all moments in the plastic range, the sum of the positive and negative resisting moments along the failure planes will be $\Sigma(M_m + M_s)$. Comparing this total moment with static resisting moments obtained by other methods, the results are as follows:

(a) Assuming a square slab clamped on all edges, the maximum theoretical stress (24) for loads within the elastic range occurs at the mid point of the side as listed in line A in figure 2.4.5-5d.

---

(24) T. Evans <u>Moment Deflection Values for a Clamped Rectangular Plate</u>

[redacted]

(b) Experimental data on square slabs indicate that stress redistribution will occur within acceptable strains. The American Concrete Institute Building Code (25) allows for such redistribution by recommending empirical coefficients as listed in line B of figure 2.4.5-5d.

(c) The use of the plastic behavior described in the derivation of the dynamic formulae will assume still greater redistribution, however, if the slab steel is unbalanced by reducing the moment resistance in the column strip by 1/3 and adding this strength to the middle strip. Then the strength provided in the middle strip will be represented by line C of figure 2.4.5-5d.

Analysis		Moment Coefficient, C (M = C x WL)			
		Middle of Edges	Center of Slab	$\Sigma M_s + M_m$	Remarks
A	Elastic	0.051	0.022	0.073	Max. at midpoint
B	A.C.I. Building Code	0.033	0.025	0.058	Middle strip
		0.028	0.021	0.049	Av. over width L
C	Limit Theory	0.031	0.031	0.062	Middle strip
		0.021	0.021	0.042	Av. over width L

FIG. 2.4.5-5d

The suggested procedure (c) results in a fairly rapid step-by-step method of analysis for slabs under dynamic load, and will provide approximately the same static strength and a similar distribution of strength as recommended in the code provisions of the American Concrete Institute. This arbitrary limitation on stress distribution may be too conservative for the design of slabs expected to show appreciable plastic deformations. For this reason several test panels have been detailed with varying ratios of strength for the middle strip versus the column strip. The limits should be reviewed as soon as additional data from the post-test analysis is available.

6. Two-Way Rectangular Panels

Rectangular panels may be designed for a given deflection under dynamic loads in a similar manner. See figure 2.4.5-6.

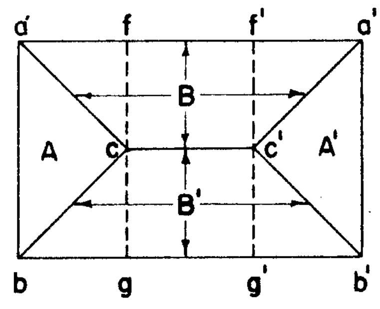


FIG. 2.4.5-6

(25) American Concrete Institute, Building Code Requirements for Reinforced Concrete (A.C.I. 318-47)

[redacted]

Panel a a' b' b may be considered as consisting of elements A, A', B, B' bordered on all edges by failure planes. If we isolate the triangular areas A and A', the dynamic deflections of points c and c' may be found by the method described previously for a square panel. The next step is to supply just sufficient moment resistance for the areas B and B' to make the deflection of line c c' compatible with the deflection computed for points c and c' of triangles A and A'.

The computation may be simplified if sections B and B' are further subdivided into triangular elements such as afc and rectangular elements such as c f f' c'. The triangular subdivisions will be satisfactory if their moment resistance per unit length along the edges is the same as for the corresponding edges of triangles A and A'.

The rectangular section c f f' c' may be analyzed as previously described for one way slabs.

The previous discussion has been concerned with a slab continuous on all edges. However, the method lends itself to various other conditions of support, so long as the failure planes can be accurately predicted. This is most readily accomplished by following the lines of maximum stress as determined from an elastic analysis.

7. Flat Slabs

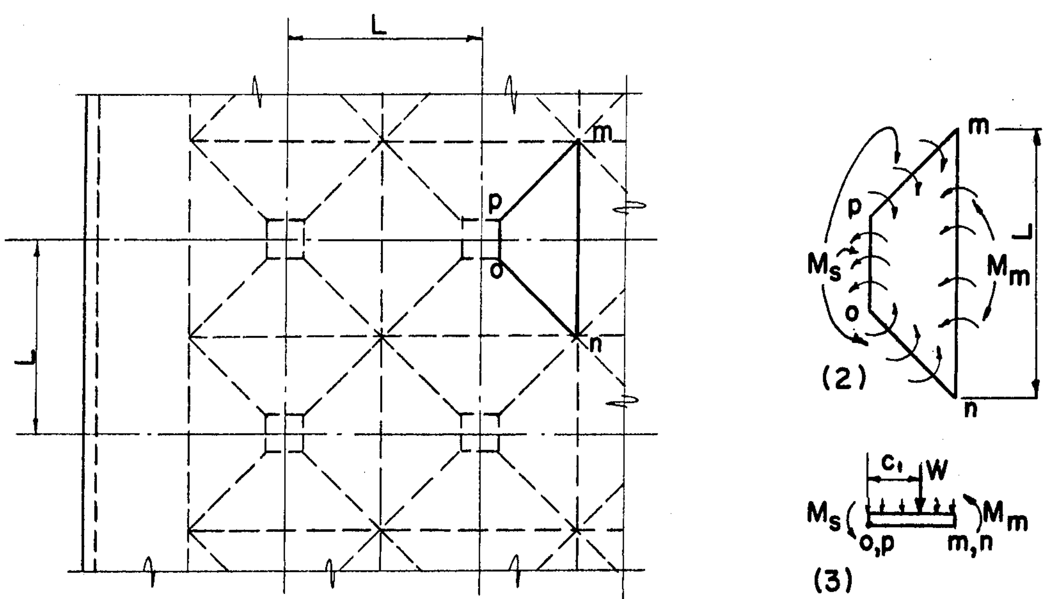


FIG.2.4.5-7

The design of flat slab panels subjected to dynamic loads may be approximated using the same approach as was adopted for the two way slabs of the preceding paragraphs.

The assumed failure planes for this framing are shown in Fig. 2.4.5-7. For equal spans under uniform loading the adjoining planes are symmetrical, the deflections are equal, and the shear between the elements and normal to the surface of the slab is zero. If shears in the plane of the slab and twisting moments along the planes of failure are ignored, each segment, such as m n o p may be separated and treated as an independent member rotating about the line of support, o p which represents the face of the column, the edge of the capital, or the drop panel.

The moment tending to rotate the member about its support will depend on the total load and the location of the lever arm as determined by the center of gravity of the load. The dynamic motion of the member will be resisted by the moments developed along the failure planes and will be influenced by the inertia forces developed by the equivalent mass of the element.

Using the constants below, a step by step analysis may proceed on the same basis as that shown in the table of figure 2.4.5-9, for a one way slab or beam. The derivation of the required constants is shown in section 2.4.5-A8.

If each segment is assumed to be supported at the center of a column, the total computed static moment will be:

$$M = 0.0833WL$$

where $W = wL^2$

and L = center to center spacing of supports

This compares with the value of 0.09 which is specified by the American Concrete Institute[25] as the coefficient of WL for the computation of the total design moment in any flat slab panel. The computed steel areas may be distributed to column strips and middle strips in the same proportions as specified by conventional code requirements.

A half-panel strip supported at the outer end by spandrel beams or by exterior walls may be considered as a member spanning in one direction. This part may be designed as a one-way slab, as described in section 2.4.5-A2.

(25) American Concrete Institute, Building Code Requirements for Reinforced Concrete (A.C.I. 318-47)

8. Constants for Segments of Any Shape

Where failure planes break a slab into irregularly shaped segments, the constants required for analysis of the segment by the method of section 2.4.5-9 may be derived as shown below.

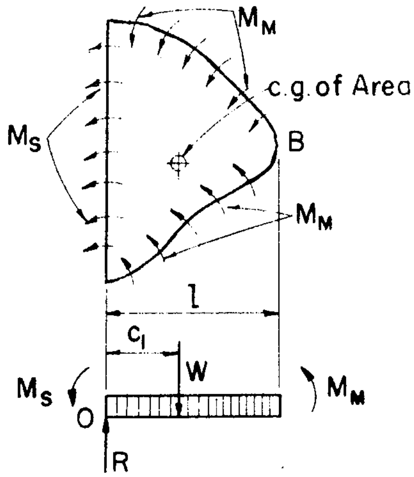


FIG. 2.4.5-8a

If a segment is supported along line 0-0 with irregular failure planes as shown in figure 2.4.5-8a, yield point moments are developed along these failure planes as indicated, and if the shear along these failure planes, normal to the surface of the slab is zero, the segment will rotate about line 0-0 under the action of the unbalanced load $(W-R)$. With notation as used previously or as indicated, the equation of motion for the segment is:

$$T = I_0 \ddot{\theta}$$

$$(W-R) \cdot c_1 = I_0 \ddot{\theta} = I_0 \frac{\ddot{x}}{l} \qquad \text{where. } x \text{ is measured at B}$$

$$(W-R) = \frac{m \rho^2}{c_1} \cdot \frac{\ddot{x}}{l} \qquad \text{where} \quad \rho^2 = \frac{I_0}{m}$$

m = mass of the segment

$$(W-R) = m_e \ddot{x} \qquad \text{Where} \qquad m_e = m \cdot \frac{\rho^2}{c_1 l}$$

Therefore, the Load - Mass factor becomes $\dfrac{\rho^2}{c_1 l}$

The supports of the segment along line 0-0 will receive a load equal to the static moment resistance of the segment, plus a dynamic reaction resulting from the rotation. This dynamic reaction may be derived, using the same notation as was used for the rectangular members or as indicated in figure 2.4.5-8b.

[redacted]

$$F_1 + F_2 = W - R$$

$$F_1 = m\ddot{x}$$

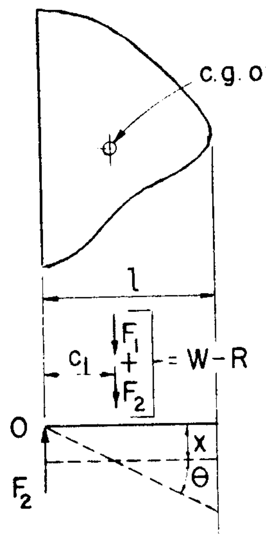


FIG. 2.4.5-8b

$$T = F_2 \cdot c_1 = I_C \ddot{\theta} \;;\; \ddot{\theta} = \frac{\ddot{x}}{c_1}$$

$$= \frac{I_C \ddot{x}}{c_1}$$

$$\therefore F_2 = \frac{I_C \ddot{x}}{c_1^2}$$

$$\frac{F_1}{F_2} = \frac{m\ddot{x}}{I_C \dfrac{\ddot{x}}{c_1^2}} = \frac{mc_1^2}{I_C}$$

$$F_1 = F_2 \left(\frac{mc_1^2}{I_C}\right)$$

$$\therefore F_2 \cdot \left(1 + \frac{mc_1^2}{I_C}\right) = W - R$$

$$F_2 = \left(\frac{1}{1 + \dfrac{mc_1^2}{I_C}}\right) \cdot (W - R)$$

Therefore the total reaction S at line 0–0 $= R + F_2$

or:

$$S = R + \frac{1}{1 + \dfrac{mc_1^2}{I_C}} (W - R)$$

9. Step-by-Step Analysis for Acceleration, Velocity and Deflection

The accelerations, velocities, and deflections of any flexural member may now be computed on the basis of the applied loads and an appropriate resistance function. The following step-by-step procedure illustrates the general procedure of such computation:

a. Select time stations, t_n , such that $t_n - t_{n-1}$ gives a time interval, Δt , consistent with the accuracy of the solution.

b. Find the average load W for the various time intervals.

c. From the resistance function find the average resistance in the given time intervals. When the resistance is a function of the displacement, the magnitude of resistance in each time interval must be computed by trial and error.

[redacted]

d. Subtract the average resistance from the average load to obtain the unbalanced dynamic load.

e. Divide the unbalanced dynamic load by the equivalent mass to obtain the average acceleration.

f. Multiply the average acceleration by the time interval to obtain the change in velocity in that time interval.

g. Sum the increments of velocity prior to any given time, t_n , to get the instantaneous velocity, v_n .

h. Compute the average velocity in each time increment as $\bar{v} = \frac{v_n + v_{n-1}}{2}$.

i. Multiply the average velocity by the time interval to obtain the change in deflection in that time interval.

j. Sum all the increments of deflection to obtain the total deflection.

A tabular arrangement for the above computation is shown in Figure 2.4.5-9 . The computation is completed at time, t_f , when the positive velocity is reduced to zero and the maximum-positive deflection is achieved.

Where only the final deflection is required the table may be greatly condensed, as shown in the table of Figure 2.5.3-1.

t	Δt	W	W_R	$W-W_R$	$\ddot{x}$	$\Delta v = \ddot{x}\Delta t$	Σv	$\bar{v}$	Δx	x
t_0							0			0
	—	—	—	—	—	—		—	—	
t_1							—			—
.										
.										
.										
t_{f-1}							—			—
	—	—	—	—	—	—		—	—	
t_f [1]							0			—
Time	Time Increments $(t_n - t_{n-1})$	W = Total Load	W_R = Total Resistance	$W-W_R$ = Unbalanced Impulse	$\ddot{x} = \frac{\Delta v}{\Delta t} = \frac{W-W_R}{m_e}$	Change in Velocity During interval $= \ddot{x}\Delta t$	$v_n = v_{n-1} + \Delta v$	$\bar{v} = \frac{(v_n + v_{n-1})}{2}$	$\Delta x = \bar{v}\cdot\Delta t$	$x_n = x_{n-1} + \Delta x$

1. Analysis completed at t_f

FIG. 2.4.5-9

[redacted]

10. Semi-graphical Solutions

The empirical resistance function derived in Section 2.4.3 can be adapted to a semi-graphical procedure for determining accelerations, velocities, and deflections. For example, in figure 2.4.5-10a the difference between the impulse and the resistance curves at any time divided by the equivalent mass, m_e, equals the instantaneous acceleration at that time.

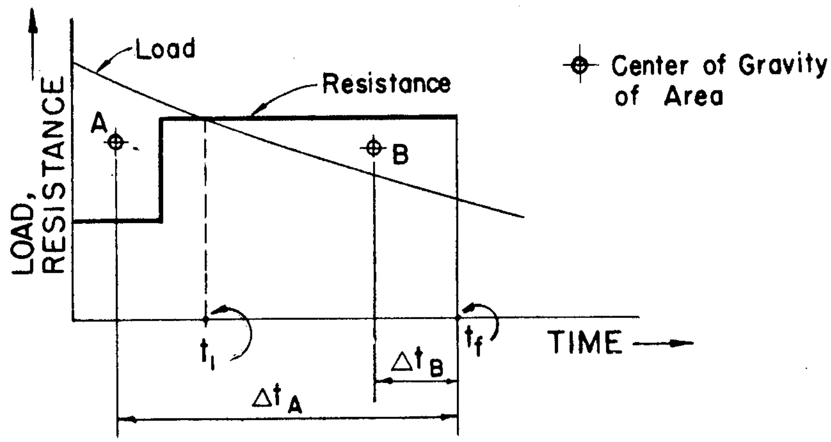


FIG. 2.4.5-10a

If we consider area A to be positive and area B to be negative, the summation of area up to any time divided by the equivalent mass equals the instantaneous velocity at that time. The maximum positive velocity occurs at time t_1.

At time, t_f, the areas A and B being equal, their sum and hence the velocity, is zero. This point also marks the maximum positive deflection of the element which may be computed as follows:

$$\delta_{max} = \frac{A(\Delta t_A) - B(\Delta t_B)}{m}$$

The deflection at any other time may be computed in a similar manner.

To be practical, for graphical solutions based on the linear stress-strain resistance functions, a linear stress-time relation must be assumed. This transformation is subject to variable degrees of error.

Figure 2.4.5-10b shows the actual response for members of different natural frequencies compared to the time of rise of the rapidly applied load.

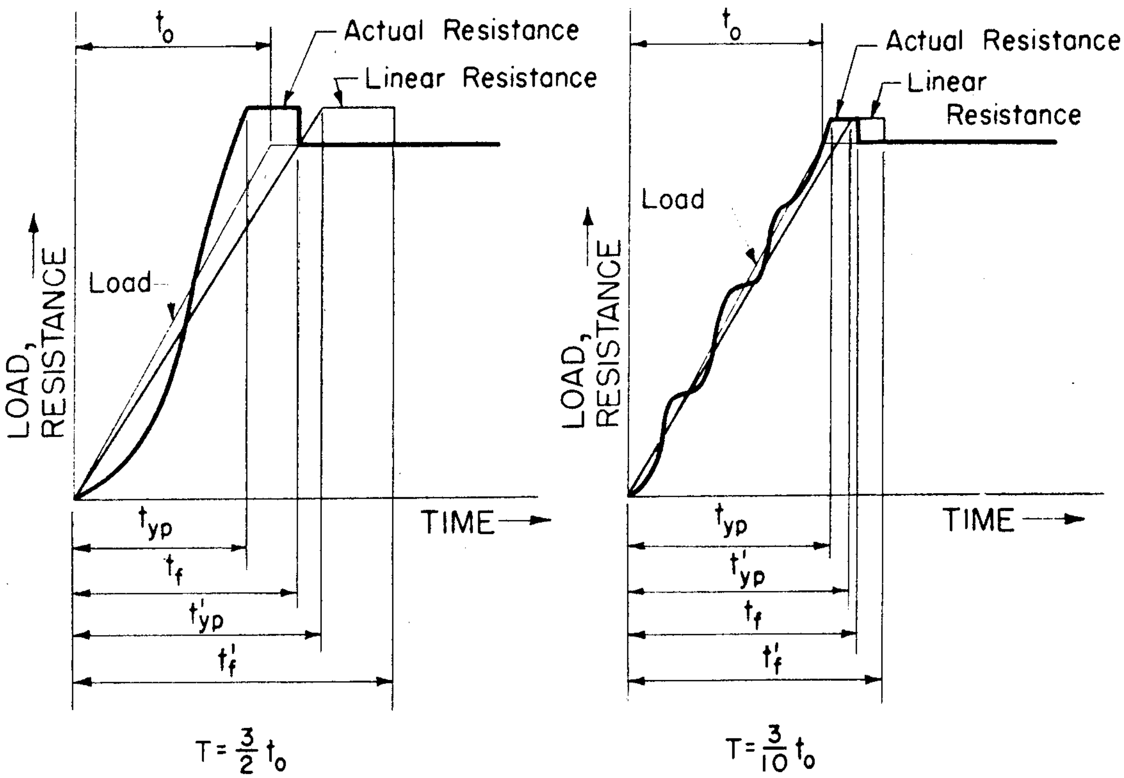


FIG. 2.4.5-10b

In the graphical solution the slope of the linear elastic resistance function is chosen to give the actual deflection at the time the yield point is reached. This is accomplished by two or more trials using the moment of areas between the load and resistance curves in a similar manner to the deflection computations for the stepped resistance function described previously.

For example, the moment of the unbalanced area between the load and the resistance curve up to the assumed yield point multiplied by the arm from the yield point to the center of gravity of this area should be equal to the yield point deflections of the assumed member multiplied by the effective mass. If the computed and assumed deflections are not equal a new slope must be tried and the process repeated. When the deflections agree the assumed slope is correct for the assumed resistance.

[redacted]

It can be seen that the time to reach the yield point, t_{yp} and t_{yp} for the actual and assumed resistance curves are different, the magnitude of error depending on the shape of the actual response curve. A similar conclusion may be made for the time to reach zero velocity, t_f and t'_f for the actual and assumed responses. Since the deflection depends on the moment of impulse areas, the final deflection using the linear response curve is also in error. Therefore, the feasibility of the graphical procedures using straight line resistance functions will depend on the allowable error. The procedure for the beam shown in figure 2.4.3-21, section 2.4.3-I is illustrated in figure 2.4.5-10c.

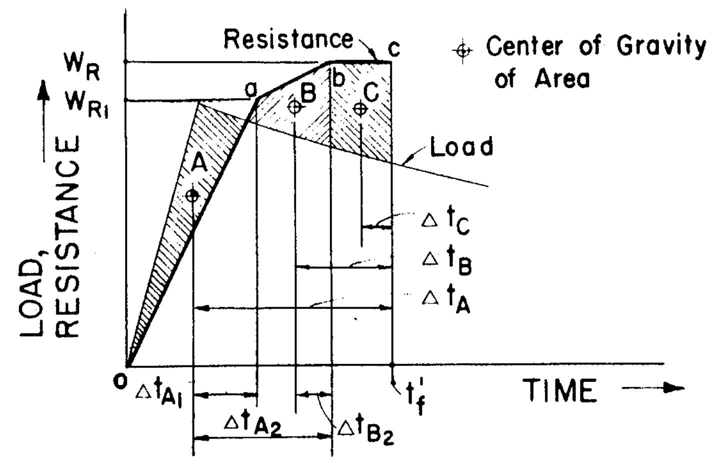


FIG. 2.4.5-10c

The slope of oa is determined from the condition, $x_1 = \dfrac{A \cdot \Delta t_{A1}}{m}$ where x_1 is the deflection at which the first yield point is reached in the member. The slope of ab is determined from the condition, $x = \dfrac{A \cdot \Delta t_{A2} - B \cdot \Delta t_{B2}}{m}$, where x is the deflection at which the second yield point is reached.

For the particular member shown the resistance is constant after the second yield point is reached. The member furnishes this resistance until the time t'_f at which time the velocity is zero $(A = B + C)$

The maximum deflection at this time is,

$$x_{max} = \frac{A \cdot \Delta t_A - B \cdot \Delta t_B - C \cdot \Delta t_C}{m}$$

B. Members Resisting Loads by Combined Shear and Direct Stress

Members, such as floor and roof slabs transmit the horizontal wall reactions to the frames or shear walls by means of direct stress and shear in the plane of the slab. In the case of the shear walls the horizontal floor reactions are distributed to the footings in the same manner.

The simplest approach to the solution of this problem appears to be an analysis in two separate and distinct steps. First the stress distribution is determined assuming the blast pressure to act as a static load, then, as a second step, the effect of the dynamic nature of the loading is considered and such corrections or adjustments are made to the results of the static analysis as may be deemed necessary.

1. Static Loading

If a flat, rigid, homogeneous member such as a plate is subjected to a load in the plane of and along the front edge of the plate and the stresses developed are confined within the elastic limit, it is possible to determine the distribution of the shearing stresses within the member by formal methods according to the theory of elasticity.

In cases where the ratio of slab depth to width is greater than one, the stress distribution may be approximated by the following simplified analysis.

Referring to figure 2.4.5-11, it is assumed that each element undergoing distortion as indicated, will move in the direction of the load until sufficient direct and shearing forces are developed to maintain equilibrium.

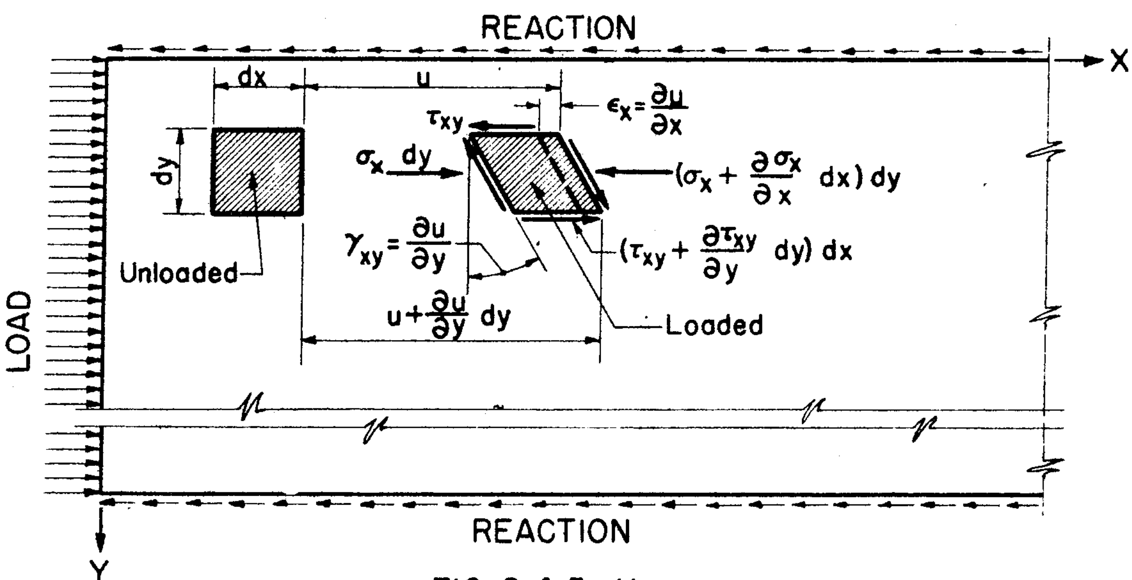


FIG. 2.4.5-11

Assuming the element to be of unit thickness and summing the forces in the X-direction:

$$\sigma_x dy - \left(\sigma_x + \frac{d\sigma_x}{dx}dx\right)dy + \left(\tau_{xy} + \frac{d\tau_{xy}}{dy}dy\right)dx - \tau_{xy}\,dx = 0$$

or

$$\frac{d\sigma_x}{dx} + \frac{d\tau_{xy}}{dy} = 0 \qquad \text{equation 2.4.5-B1}$$

For the elastic case the strain relationships are as follows:

$$E = \frac{\sigma_x}{\epsilon_x} \qquad \text{and} \qquad G = \frac{\tau_{xy}}{\gamma_{xy}}$$

Therefore:

$$\frac{d\sigma_x}{dx} = E\frac{d^2u}{dx^2} \qquad \text{equation 2.4.5-B2}$$

and

$$\frac{d\tau_{xy}}{dy} = G\frac{d^2u}{dy^2} \qquad \text{equation 2.4.5-B3}$$

Substituting (2) and (3) in (1) and letting $n^2 = \frac{G}{E}$

$$\frac{d^2u}{dx^2} + n^2\frac{d^2u}{dy^2} = 0 \qquad \text{equation 2.4.5-B4}$$

The solution of equation 2.4.5-B4 on the preceding page, in accordance with the assumed boundary conditions, provides the stress distribution curve shown in figure 2.4.5-12.

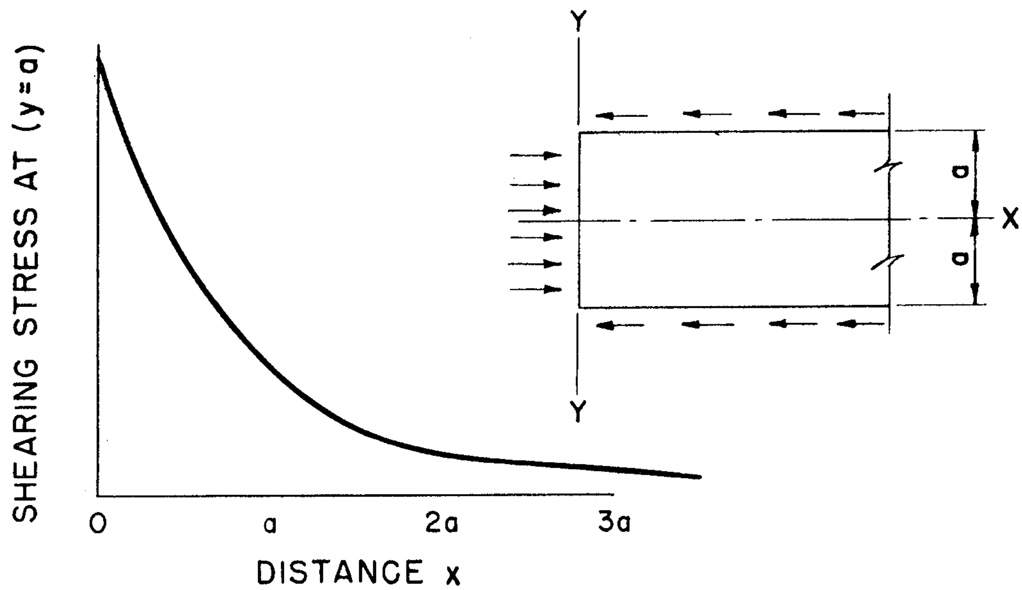


FIG. 2.4.5-12

The steel in walls and slabs is usually uniformly dispersed throughout the member and is not ideally located to carry such concentrations as indicated. If, however, the member is designed for strains above the elastic limit, then the plastic deformation will give a more uniform distribution of stress and more of the uniformly distributed reinforcing steel will be stressed to the yield point, thus making for more efficient use of the steel. However, assumptions of plastic deformation should be investigated to find their effect on buckling stability. The deformations should not be so large as to prevent the use of the building.

For the design of the test structures the finite difference method recommended by D. McHenry (26) (27) was adopted and was used to compute strains and stresses within the member. This method may be described briefly as follows:

The member being investigated is replaced by a grid dividing the section into small elements; the grid being composed of an equivalent statically indeterminate truss with square panels and counters. The elastic relationship between the motion of each point on the grid and the associated or connected points on the grid is established, after which each point is relaxed or allowed to displace under the given loads and boundary con-

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(26) D. McHenry, "A Lattice Analogy for the Solution of Stress Problems"

(27) D. McHenry, "Lattice Analogy in Concrete Design"

[redacted]

ditions. By means of a systematic numerical procedure, the effect of the displacement on all other points is duly recorded. When all points have been relaxed and the entire system is in equilibrium, the solution has been accomplished. The stresses may be easily calculated from the final displacements. A detailed example showing the use of this method is presented in Appendix 5 .

The method is quite flexible, permitting studies of action involving changes in behavior of the material with change in strain. Probably one of the most important attributes in this respect is that load redistribution at stresses beyond the yield point may be effectively estimated either by changing the stress-strain relationships between adjacent points as their respective yield points are reached or by redistributing stresses which exceed the yield point.

2. Dynamic Loading

When a large heavy member is subjected to dynamic loads, the particles are accelerated as the load is applied then decelerated as the momentum of the mass is reduced by the reactions. While the member is accelerating, its resistance to strain and the force developed due to the inertia of the particles will combine to resist the applied load. When it starts to decelerate, the required strength will depend on the combination of the force due to the momentum of the particles as well as the magnitude of the load still being applied.

Assuming a rectangular pulse loading and unyielding supports, if the motion is stopped within the elastic range of the resisting materials, the rate of acceleration and deceleration will be equal and the unit stress will be twice that obtained under the same static load. If, for the same loading, the strain is allowed to exceed the elastic limit, several interdependent actions occur which permit further reduction in strength. By reducing the design strength in order to allow the strain to exceed yield point strain, the net excess of resistance over load is decreased, thereby causing the mass to decelerate at a slower rate and reducing the total force causing motion (which at this time is composed of the applied load plus the inertia force of the decelerating mass.) Furthermore, the peak stresses are reduced because of redistribution of the stress. The necessary design strength is therefore reduced from that required to maintain the strains within the elastic limit, because both the applied design load is lower and the stress distribution is more uniform.

Ductile, heavy mass members will be even more effective in resisting short duration peak loads than described above for rectangular pulse loading.

As shear members are inherently stronger than the flexural members of a building frame, the strength requirements for these members are not severe. In view of this, as well as in consideration of the difficulties in obtaining a reasonably accurate analysis under the necessary but doubtful assumptions, it seems preferable to consider only

[redacted]

limiting conditions, and to check these against test behavior, leaving the complete solution until more basic data is available.

3. Recommended Design Procedure

(a) Determine the stress pattern using twice the given dynamic load as an applied static load. Use the appropriate curve from figure 2.4.5-13 to determine the distribution of the applied load. If the building is of "finite" length relative to its width, use the same load distribution curve, but at the discontinuous edge apply an edge load of the same proportions compared to the load on the front edge, as the distributed shear at that point is proportional to the shear at the front edge. Superimpose the load distribution of the first and second edge loads using the proper signs.

(b) Decrease the stresses found in step (a) by a factor of 1.35 representing the expected increase in strength due to the rate of loading. This increase in strength under dynamic loads is a conservative estimate based on comparisons with the increase in strength of members under bending stress which have slower rates of loading. As mentioned previously in section 2.4.3, the strengths of members under flexure were increased approximately 35% to 60%. Using the same 35% increase in strength for the shear wall buildings will be equivalent to using 148% as the dynamic load factor rather than the 200% described in step (a).

(c) Some redistribution of stress may be arbitrarily allowed by permitting the stress at points of maximum load concentration to exceed the yield point. The permissible amount of stress redistribution will depend on the ratio of the peak stress to the average stress and on the amount of strain which may be tolerated in the structure. As an example, consider the case of the frame buildings and the shear wall building of the test structure, where the floor slabs deliver the horizontal wall reactions to the frames and shear walls respectively. In the case of the frames, the entire wall load is transmitted from the front walls to the frame through the floor. A peak stress concentration will occur at the intersection of the front edge and the frame as shown in curve 1 of figure 2.4.5-13 as well as in the preceding figure. Figure 2.4.5-13 is drawn for the case of an approximately square building.

The building with shear walls, in contrast, has part of the front wall pressure carried directly from the front wall to the shear wall due to the fact that the shear wall is continuous with and furnishes end reactions to the closure wall. The stress distribution for this case is shown in curve 2 of figure 2.4.5-13. As a first estimate, pending more information, it is recommended that the peak load be averaged over bands equal to a in length.

[redacted]

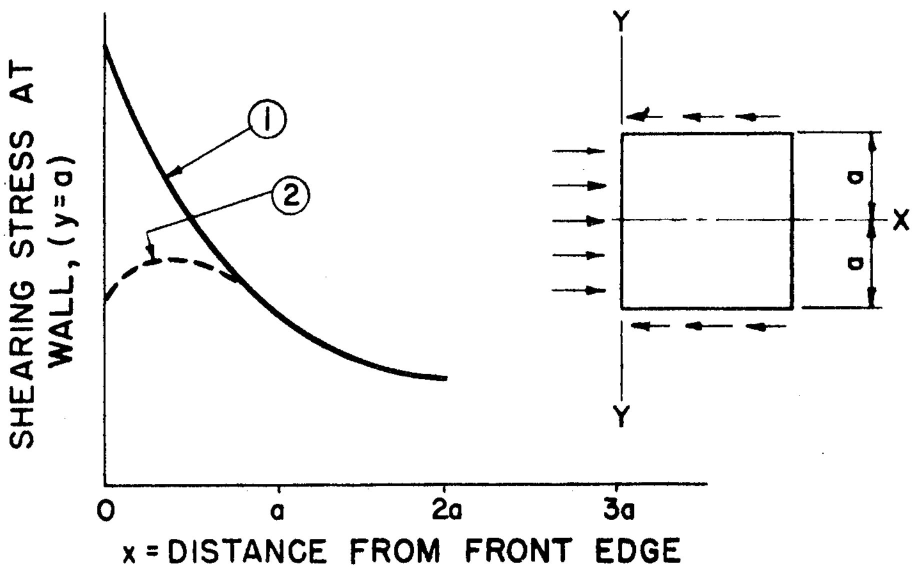

FIG. 2.4.5-13

4. Buckling Stability of the Floors

The elastic stability of the floor slab should also be investigated for combined direct and shearing stresses.

Assuming that the floor slab is homogeneous in action, then the critical buckling load may be assumed to be of the form $\sigma_{cr} = \dfrac{k\pi^2 E h^2}{12(1-v^2)b^2}$ similar to the criteria for flat plates under static loading. As the requirements are not usually high, it is probable that use of the static load condition will be satisfactory providing a reasonable factor of safety, such as 4, is maintained. Of the remaing factors, E may be taken as 4,000,000 p.s.i. under the short time dynamic loads; b is the width between stiffeners or girders, h is the slab thickness, and k will depend on the conditions of edge support which is usually given in terms of the ratio of "A," the distance between stiffeners or reactions and "b", the width between the stiffeners.

The slab may be checked using a general value of $k = 4$. If the slab appears low in strength using this value a more accurate investigation may be made in accordance with investigations of similar problems (13),(14), (15). As the stability varies with the square of the slab thickness, large increases in strength may be obtained by small changes in the steel percentages and in the type of framing, so great accuracy is not necessary. The linear relationship assuming that the sum of the ratios of the bending stress to the ultimate stress plus the direct stress to the buckling stress is equal to or less than one is probably sufficiently accurate for the condition of combined bending and direct or shearing stresses described above.

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(13) S. Timoshenko, Theory of Elastic Stability

(14) E. E. Sechler & L. G. Dunn, Airplane Structural Analysis and Design

(15) J. E. Younger, Mechanics of Aircraft Structures

[redacted]

This may be expressed analytically as follows:

$$\frac{P/A}{\sigma_{cr.}} + \frac{M/Z}{\sigma} \leq 1$$

where P = Total edge load

A = Cross-sectional area of the slab

M = Bending moment in a direction perpendicular to the front edge

Z = Section modulus of the plate

When the bending stress occurs in direction parallel to the front edge, it is unnecessary to combine the stresses in the manner shown above. For this condition the curvature due to bending tends to increase the stability of the slab.

For a combination of shear and bending the design should be based on the principal stresses in the slab. As concrete is strong in direct shear but weak in tension, the principal tensile stress is usually the governing factor in design.

C. Analysis of Building Frames Resisting Lateral Loads in Flexure

1. Steel Rigid Frames

a. Lateral Resistance

(1) General

As lateral loads are applied to a building frame relative horizontal displacements take place among the various floors. As a result the columns, whose extremities are restrained against rotation, develop end moments and horizontal shearing forces which oppose the displacement.

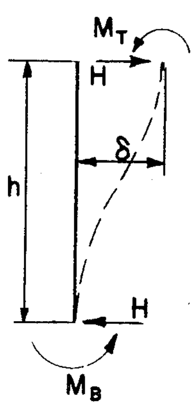

FIG. 2.4.5-14

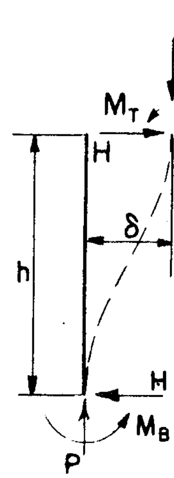

FIG. 2.4.5-15

[redacted]

From the figure 2.4.5-14 it can be seen that the magnitude of the opposing shear in any member will be

$$H = \frac{M_T + M_B}{h} \qquad \text{Equation 2.4.5 C-1}$$

For any given value of δ, specific values for M_T and M_B may be found from the resistance function of the column as a structural member under combined bending and direct stress. If there is no axial load then $P \cdot \delta$ will be zero and equation 2.4.5C-1 will apply. In this case the specific values of M_T and M_B may be found directly from the relative deflection of the ends and the idealized resistance curve described in Section 2.4.3., and shown again in figure 2.4.5-16.

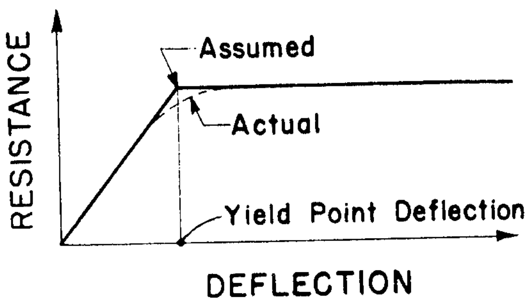

FIG. 2.4.5-16

If the member is subjected to an axial load P as well as a lateral displacement, as shown in figure 2.4.5-15, the lateral resistance is reduced by the amount $\frac{P \cdot \delta}{h}$ or $H = \frac{M_T + M_B - P \cdot \delta}{h}$ Equation 2.4.5 C-2

(2) <u>Combined Bending and Axial Load - Plastic Range</u>

In the plastic range, which embraces the greater part of the motion, the axial load bending moment relationship may be obtained as described in sections 2.4.3-E and as shown in figure 2.4.5-17, curve "a".

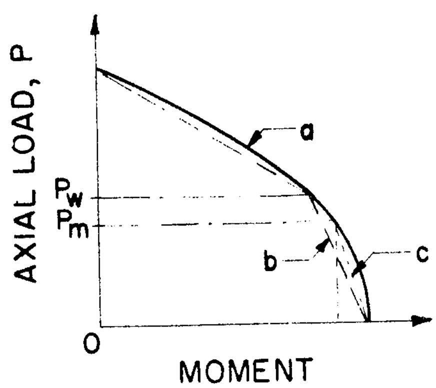

FIG. 2.4.5-17

[redacted]

A large number of points are required to define accurately the theoretical load-moment curve a . In actual design, however, straight lines may be drawn from the point of full plastic moment without axial load M at $P=0$ to a second point in which the full web area is required for axial load M at P_w and continued to a third point in which the entire section is required to carry the axial load $M=0$. The broken line defined by these three points is sufficiently accurate for practical purposes. If the maximum axial load is comparatively small, (as P_m in figure 2.4.5-17), the error in using the constant resistant moment M at $P=0$ will not exceed $(M \text{ at } P=0)-(M \text{ at } P_m)$. Then a constant moment-resistance curve similar to curve "c" of figure 2.4.5-17 with the M_{ult} as some average value between M at $P=0$ and M at P_w may be used with reasonable accuracy and with an appreciable reduction in design effort.

(3) Combined Bending and Axial Load - Elastic Range

The moment-resistance curve in the elastic range will depend on the relative importance of such factors as the flexural and shearing strains in the column, the slip in riveted connections, and the magnitude of the joint rotation.

b. Flexure in Columns

Below the yield point, the contribution of the flexural strain to the total deflection may be computed by the usual formulae based on the elastic theory. As the elastic deflection and the elastic curvature are functions of the moments along the member, the moment due to the eccentricity of the axial load must also be considered. However, if the axial load is small (as P_m), the moment due to its eccentricity will also be small and the effect on the elastic curvature will be small.

c. Shearing Strain in Columns

The deflection due to shearing stresses may also be computed by the usual formulae for stresses within the elastic range. An approximate expression for this component of the deflection would be

$$\delta_s = \frac{H \cdot h}{A \cdot E_s}$$

Where δ_s = Lateral deflection due to shear strain

H = Horizontal shearing force

h = The clear height of the column

A = Area of the web (assuming a WF member)

E_s = The modulus of elasticity in shear

As shown by equations 2.4.5-C1 and C2, H is also a function of the moment and hence of the axial load. If the shearing stresses in the vicinity of the plastic hinges exceed the elastic limit, the deflection due to shearing deformations will increase. The high stress, however, would be confined to a small length Δh and probably would not have much influence on the total deflection.

d. Slip in Joint and Connections

The slip and deformations in the connections will depend to a large extent on the type of beam to column connections used in the structure. Welded connections with adequate stiffeners contribute very little to the total deflection. Riveted connections, on the other hand, permit local bending of the column flanges and connecting material, extension of the tension rivets, and slip and local distortion of the rivets in shear.

Several investigations have been made of the deformation and slip of riveted connections (28),(29). In view of these test results, it seems reasonable to assume a joint rotation of from 0.001 to 0.003 radians, (depending on the depth of the connecting members) if the web stiffeners and tension rivets are designed to be elastic. This can be only an approximate figure until further tests more closely approach the conditions met in actual structures. Figure 2.4.5-18 illustrates the difference in loading conditions in the usual moment connection test setup, and the conditions which will occur in the test frames.

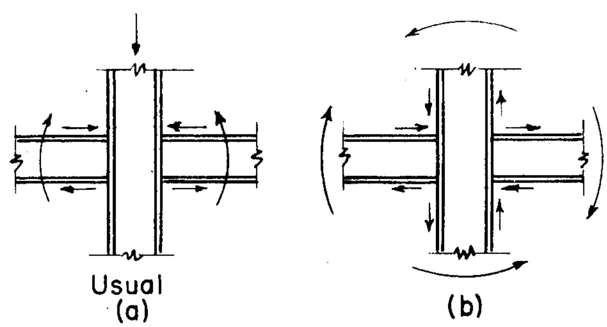

FIG. 2.4.5-18

In the preliminary studies, based on riveted Tee connections, it was assumed that all joint rotation was elastic. This seems to be a reasonable assumption as the actual $M - \theta$ diagrams obtained

(28) "2nd Report of the Steel Structures Research Committee", Dept. of Scientific and Industrial Research

(29) R. A. Hechtman and B. G. Johnston, "Riveted Semi-Rigid Beam-to-Column Building Connections"

[redacted]

by test of similar moment connections (28),(29) are of the form shown in figure 2.4.5-19. The joints, even with unstiffened column webs and flanges, showed very high rigidity up to rivet stresses of approximately 30,000 p.s.i. Slip of the shear rivets appears to be practically the only factor causing non-elastic behavior in this type of connection which cannot be neglected.

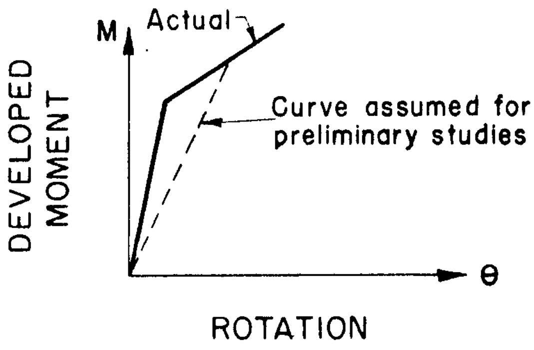

FIG. 2.4.5-19

6. <u>Flexure in Floor Beams</u>

The rotation of the column ends due to flexural deformation of the connecting floor girders could be handled by rigorous methods of elastic analysis. However, the contribution from this source to the total elastic deflection is very small and it was felt that such an analysis was not justified.

This is particularly true where concrete floor slabs are used with the steel girders. In this case shear clips are used to transfer the horizontal loads from the slab masses to the building frame and vice versa. The shear clips in turn create a sufficiently strong bond between the floor slab and girder to make them act as a single composite section. As the girder itself must be strong enough to develop the necessary column restraint at the connection, the composite section away from the connection will have a very high rigidity and the angular rotations of the girder ends will be small.

It is therefore sufficiently accurate to compute values of the θ for the various girders using the maximum column moments, and then assuming that the average θ (top and bottom) for any other column moment will vary directly as the ratio of the actual moment to the maximum moment. This consideration, substantiated by comparative studies, indicates that the error caused by this assumption, shown in figure 2.4.5-20 is negligible.

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(28) "2nd Report of the Steel Structures Research Committee", Dept. of Scientific and Industrial Research.

(29) R. A. Hechtman and B. G. Johnston, "Riveted Simi-Rigid Beam-to-Column Building Connections".

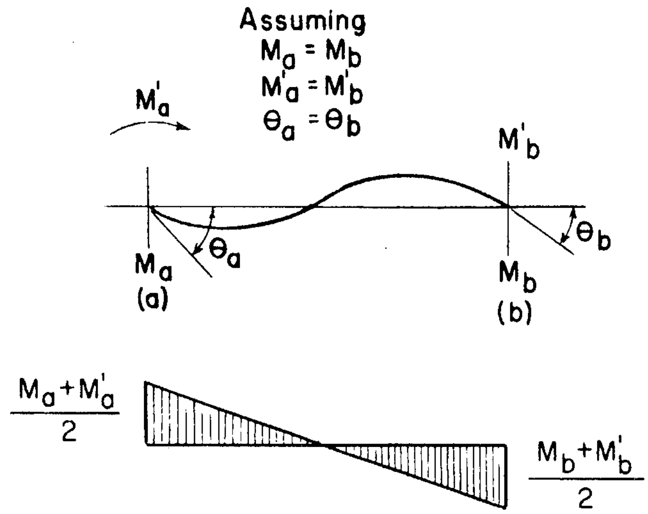


FIG. 2.4.5-20

Referring to figure 2.4.5-20, the value of θ_a and θ_b is easily obtained, given the I of the girder.

f. Recommended Procedure

Of the numerous factors listed above as contributing to the total elastic deflection of the frame, the elastic deflection of the column is, in the usual case, as great as all the other factors combined. For this reason approximations involving small errors in the other factors will not materially effect the accuracy of the analysis. This is particularly true when it is remembered that the elastic deformation occurs only for a relatively small part of the total movement.

As the corresponding end moments may be computed for any given elastic deflections, as described above, and since the yield point moment $M_{ult.}$ is a function of P, it is convenient to compute the elastic deflection δ_e for various values of $M_{ult.}$, and plot these as a function of P as shown in figure 2.4.5-21. This figure may be amplified to define completely the resistance function of a particular column.

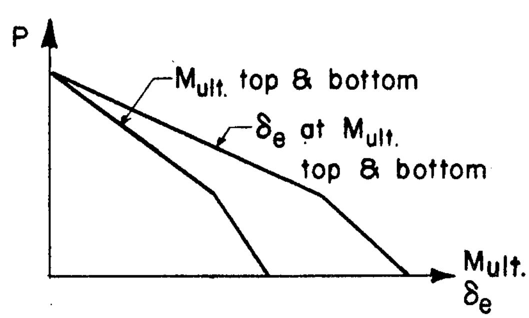


FIG. 2.4.5-21

[redacted]

Figure 2.4.5-21 shows the P-M and P-δ_e curves for the simplest possible case; a column of uniform cross section with equal joint rotation at top and bottom.

If the column is not of uniform cross section, the picture will change. In riveted construction, for example, holes in the flange at the connection of the column to the girder may result in a section unsymmetrical in action, either because of a different number of rivet holes at either the top or bottom ends, or because the holes at one end are on the tension side of the column and at the other end are at the compression side as shown in figure 2.4.5-22 for an exterior column.

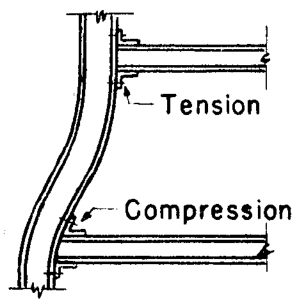


FIG. 2.4.5-22

It is assumed in this case that compression in the flange may be transmitted through the rivets, but that in tension areas the area of the rivet holes must be deducted from the gross flange area. To simplify the analysis it is arbitrarily assumed that the full cross-sectional area of the hole should be deducted if the theoretical neutral axis for the fully plastic condition passes through the rivet hole. The P-M curve (axial load versus ultimate resisting moment) for a section with rivet holes deducted is plotted as three straight lines as shown in figure 2.4.5-23.

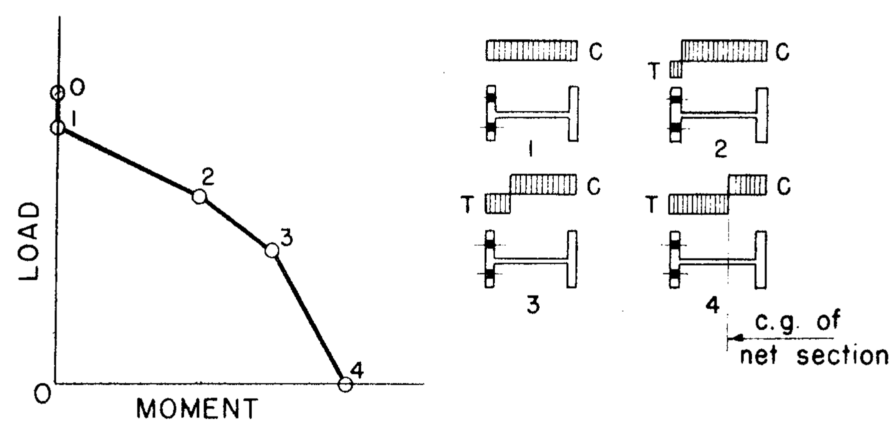


FIG. 2.4.5-23

[redacted]

at (0) $P = A_G \cdot f_y$

(1) $P = A_n \cdot f_y$

(2) $P = (A_w + A_h) f_y$

(3) $P = (A_w - A_h) f_y$

(4) $P = 0$

where A_G = Gross area of section

A_n = Net area of section

A_w = Web area

A_h = Area of holes

f_y = Allowable stress on the section

In this case the $P-M$ curves for top and bottom are as shown below in figure 2.4.5-24.

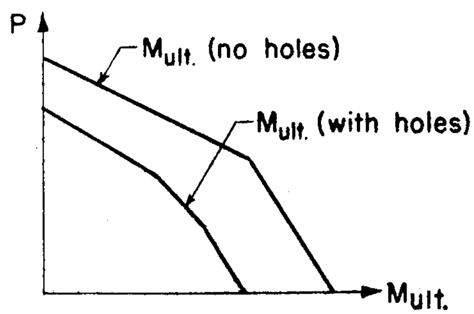


FIG. 2.4.5-24

Assuming equal rotation at top and bottom, the moments at both ends will be equal, up to the time the weak end yields, since the rivet holes have a negligible effect on the overall stiffness of the members as shown in figure 2.4.5-25.

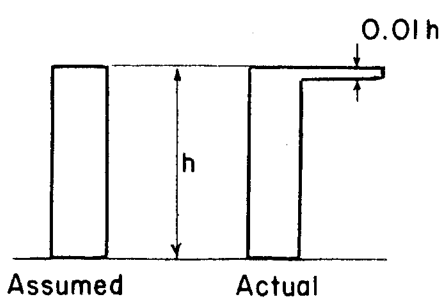


FIG. 2.4.5-25

After the yield point is reached at the weaker end, the stronger end will continue to pick up moment as the deflection continues to

increase, but at a decreased rate because of reduced stiffness due to the plastic hinge at the far end as shown in figure 2.4.5-26.

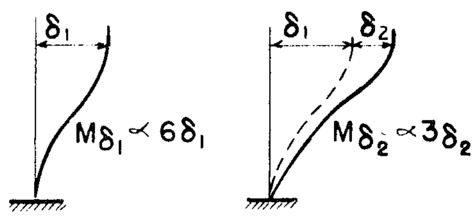


FIG. 2.4.5-26

For any given value of P, the variation of moment with deflection will be as shown in figure 2.4.5-27.

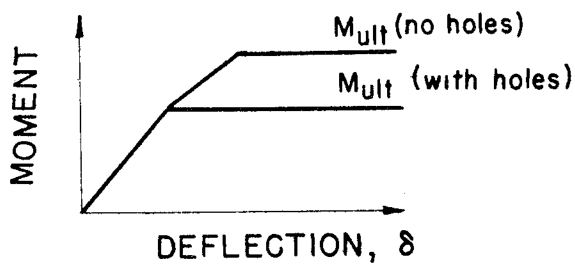


FIG. 2.4.5-27

Figure 2.4.5-28 shows the $P-M$ and $P-\delta_e$ curves for a column having holes deducted at one end only.

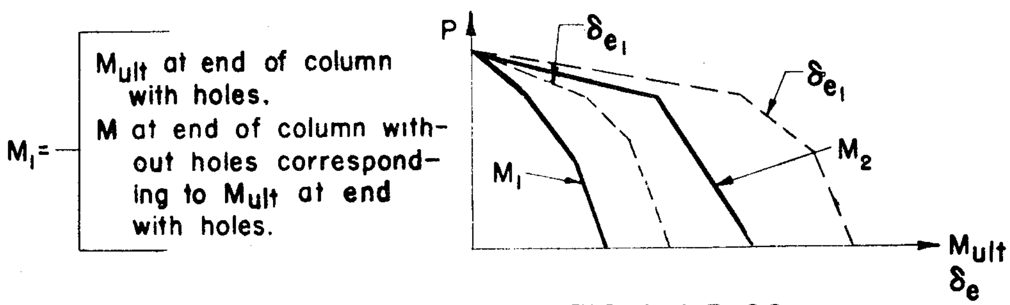


FIG. 2.4.5-28

In the case of a column with an abrupt change in cross-section, figure 2.4.5-29, the moments at the ends will, of course, be different throughout the entire range of action and the $P-M$ and the $P-\delta_e$ curves will consist of the following curves:

[redacted]

(1) $M_{ult.}$ at small end

(2) M at large end corresponding to $M_{ult.}$ at small end

(3) $M_{ult.}$ at large end

(4) δ_e corresponding to $M_{ult.}$ at small end

(5) δ_e corresponding to $M_{ult.}$ at large end

The various curves obtained from considerations (1) to (5) and the approximations given in section 2.4.5 C are indicated in figure 2.4.5-30.

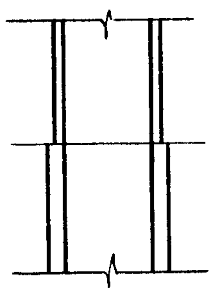


FIG. 2.4.5-29

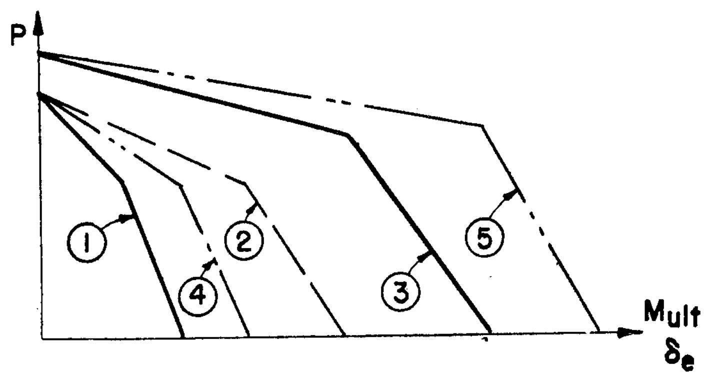


FIG. 2.4.5-30

Further complication in the analysis may be encountered if the joint rotations at either end are not the same at a given time. In this case the moments must be made compatible with the assumed conditions as shown in figure 2.4.5-31.

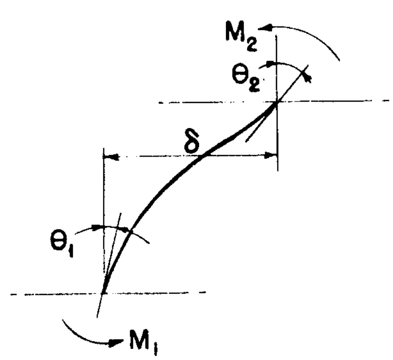


FIG. 2.4.5-31

[redacted]

g. Reversal in Direction of Loading

It should be noted that the elastic deflections plotted in the preceding sections are for the case of a column deflecting continuously in the same direction. However, if the end moments exceed the yield point and then reverse in sign, the reversal, assuming a constant P and equal strength at the top and bottom of the column, proceeds as shown in figure 2.4.5-32.

If the strength of the top and the bottom of the column differs due to rivet holes or different column strengths above and below the column splices, the action will be more complicated.

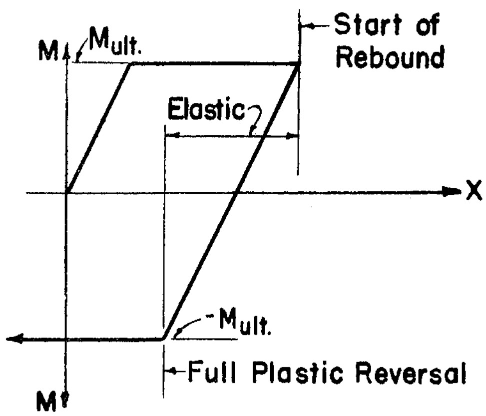


FIG. 2.4.5-32

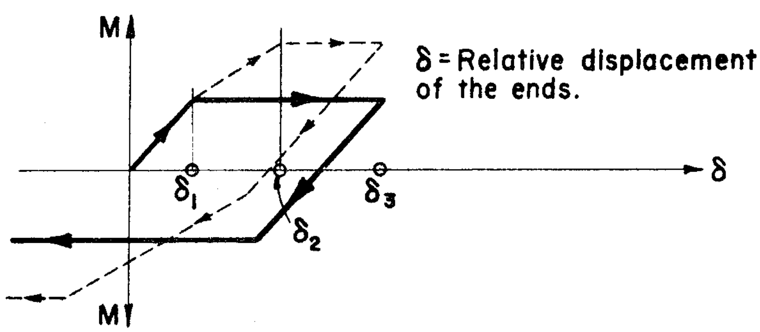


FIG. 2.4.5-33

In this case the change in moment with deflection, figure 2.4.5-33, will be similar at the two ends until one end reaches the yield point ($\delta 1$). The moment at that end will then remain constant while the moment at the other end increases at a decreased rate until that end also reaches its yield point ($\delta 2$). Assuming a constant value for the axial stress, the resistance will then remain constant until the load is reversed ($\delta 3$). At this time

[redacted]

both ends will start rebounding at the same rate. If the reversed displacement is continued the members will again become plastic but with moments of opposite sign.

The above curves are dependent on the assumption of a yield point stress for the steel as described in section 2.4.1. Given this value the $P-M$ and $P-\delta_e$ curves may be plotted as described, thus defining, within the limits of accuracy of the assumptions made, the resistance functions for the columns. From these curves, given the axial load and the relative deflection, the ultimate end moments may be determined and all quantities necessary for the computation of the horizontal resistance, $H = \dfrac{M_T + M_B - P\cdot\delta}{h}$, are available.

2. Resistance of Concrete Rigid Frames to Loads Causing Lateral Displacement

a. General Behavior

As in the case for steel frames, the resistance of the columns to displacement of one end relative to the other end will cause column shears of $H = \dfrac{M_T + M_B}{h}$ for columns without axial load or $H = \dfrac{M_T + M_B - P\cdot\delta}{h}$ for columns with axial load.

b. Moment Resisting Curve

Using the formulae presented in section 2.4.3 G, the ultimate strength of reinforced concrete columns may be computed for various loading conditions and the curve of axial load versus ultimate moment capacity can be drawn as shown in figure 2.4.5-34.

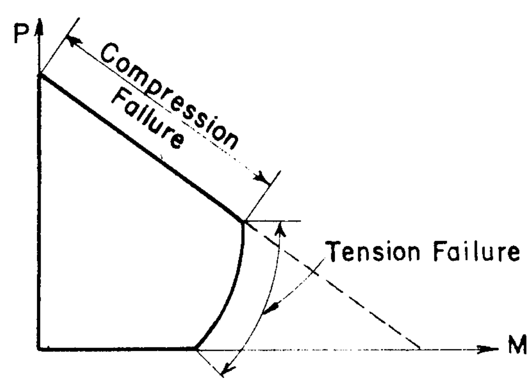


FIG. 2.4.5-34

[redacted]

For any given value of P, the M-δcurve for a column is assumed to be as in figure 2.4.5-35 where M_{ult} is obtained from figure 2.4.5-34.

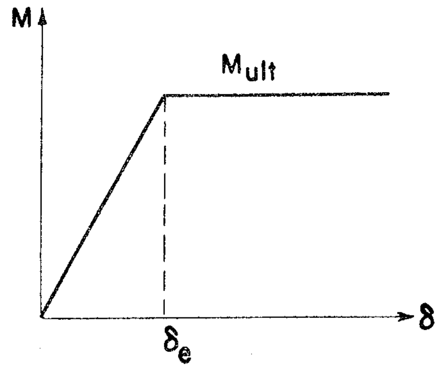


FIG. 2.4.5 -35

c. Moment Resisting Curves - Elastic Range

The elastic deflection of a concrete column, like a steel column, depends on the flexural deformation and shear distortion in the columns and the rotation of the ends due to the elastic curvature of the girder. Monolithic reinforced concrete construction, unlike the riveted steel frame, does not have joint slip, barring possible slip of the bars under high bond stress.

The flexural deformation of the concrete columns is computed on the basis of the elastic formulae $\delta_e = \frac{kMh^2}{EI}$, using $n = 10$ for the value of $\frac{E_s}{E_c}$, and a moment of inertia based on the gross cross-sectional area of the concrete plus the transformed area of steel. This method for computing the moment of inertia is not exact, but gives a reasonably accurate approximation for the overall stiffness of members carrying bending plus axial load.

As the computed displacement due to shear distortion in the concrete columns is small, it is believed that this factor may be omitted in computing the total elastic deflection. Where the columns are relatively short and deep, the shear strain may be approximated using the same formula as used for the steel columns by substituting the appropriate value for the shearing modulus.

The contribution of the girder deformation to the end rotation of the columns may be computed by the same methods as used for the steel frames. The concrete floors and girders generally are stiffer and permit less rotation than do the steel girders. The roof girder, in particular, supplies almost complete fixity to the top of the upper column.

For columns having different ultimate moments at the top and bottom, two P-M curves (figure 2.4.5-36) are required to describe the axial load-bending moment relationships.

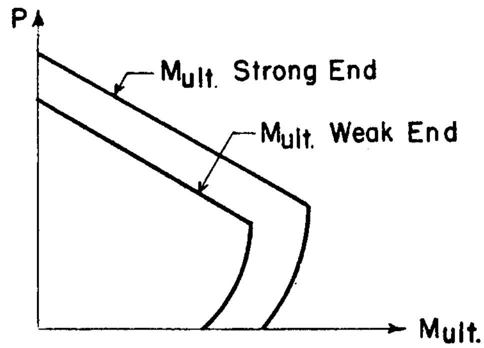


FIG. 2.4.5-36

If the elastic deflection is due to flexural strains only, curves of elastic deflection are not required, as it is a simple matter to compute a constant, C, for the equation $M = C\delta$ and compute the end moments algebraically for deflections within the elastic range. The value of C in the formula above will depend on whether both ends are fixed, or one end has already entered the plastic region. This constant can also be modified to include the effect of girder rotation, or an $M-\delta e$ curve may be plotted as in the case of the steel columns. These curves are shown in figure 2.4.5.-37.

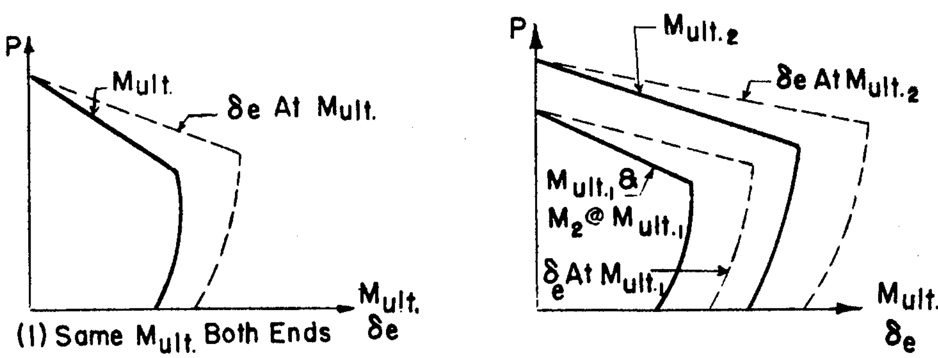


FIG. 2.4.5-37

The use of these curves is in every way similar to the curves for steel columns.

3. Method of Design Analysis (Walls Not Participating)

a. Single Story Frame

A single story frame may be idealized as shown in figure 2.4.5-38.

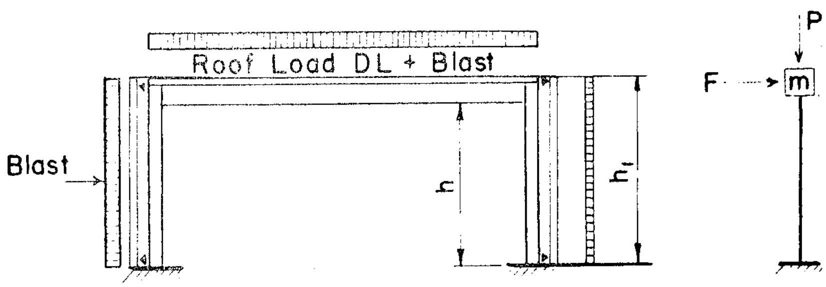


FIG. 2.4.5-38

(1) Determination of the Mass

It is assumed that the mass of the roof plus one-half of the total mass of the walls and columns are concentrated at the roof level. This is necessarily an approximation, since the wall, if hinged and rotating about the bottom, would provide an equivalent mass resisting the motion of only one-third of the total mass. It may be pertinent to note that the blast load itself has no mass, so far as the frame deflection is concerned, the inertial forces being dependent only on the masses described above.

In the usual case, however, the mass of the walls and the columns are small compared to the floor slabs and the error involved is small. In the particular instances where this ratio of masses is not true, the more accurate mass distribution should be used.

(2) General Behavior

Immediately after the impact of the blast on the structure the roof will accelerate horizontally against the inertial resistance of the roof mass and the shear in the columns. The magnitude of the resistance of the columns to relative displacement of the roof with respect to the floor will depend on the end moments which are developed. These moments, as previously described, depend on the strength and stiffness of the column and the connecting members, the rigidity of the connections, the lateral deflection, and the rate of deformation. The acceleration under the unbalanced blast force and the resisting shear acting at any instant will be numeric-

[redacted]

ally equal to $a = F/m_e$ where a is the acceleration, m_e is the equivalent mass, and F is the net force. As the value of the net unbalanced force acting on the frame is the difference between the blast load and the frame resistance, and as these depend on a number of complicated factors which are impossible to express accurately as a simple function, the step-by-step method of analysis is the most advantageous method of solution. This procedure consists primarily of simple and rapid numerical computations.

(3) Detailed Behavior

The detailed procedure for the analysis of a single story frame with self supporting walls is similar to the analysis of wall panels except for the additional consideration of the direct thrust in the column. If the direct thrust or axial load is small, its effect on the resisting moment is also small, and by neglecting the axial load the solution is identical with that previously described for beams and slabs. However, if the axial load is not small, both the value of the resisting moments at the top and bottom of the column and the portion of the resisting moments required to balance the deflection moments ($P \cdot \delta$) will be affected.

The design procedure is then as follows: At the instant the blast reaches the face of the building, the velocity and the acceleration of the mass of the building and the displacement of the roof is zero. After a finite increment of time, $t_1 - t_0$, the frame will have moved and assumed a shape similar to that shown in figure 2.4.5-39.

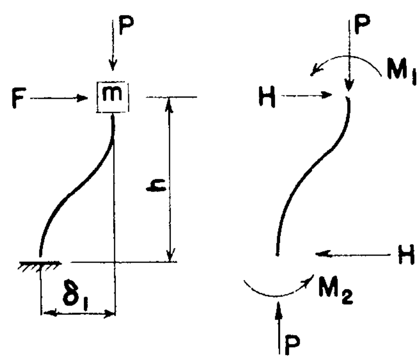


FIG. 2.4.5-39

At this time, t , end moments exist in the column as a result of the relative lateral displacement δ_1 , and the column shear resisting the applied load is

$$R = -H = -\left(\frac{M_1 + M_2 - P \cdot \delta}{h}\right)$$

For the first time increment, $t_1 - t_0$, the average resistance may be taken as $R_v = \frac{R}{2}$ since the initial resistance is zero, and the average applied load as $F_{av.} = \frac{F_0 + F_1}{2}$. The average acceleration is then equal to $a_{av.} = \frac{F_{av.} + R_{av.}}{m_e}$

The use of average values introduces a small error which can be eliminated by assuming a linear variation of the acceleration during the time interval.(30) However, such accuracy is unwarranted because neither the loading nor the resistance is known with corresponding precision. Furthermore, in the plastic state which exists for the greater part of the time of deformation, the resistance is very nearly uniform and the average value for the time increment is practically equal to the value obtained by assuming a straight line variation from t_n to t_{n-1} . In the elastic range, the accuracy of the approximation which assumes a constant average resistance between successive times may also be improved by reducing the time interval. Later discussion will include studies in which the estimated resistance is varied in many ways. The effect of various assumptions on the total computed deformation will be listed and compared.

(4) Detailed Method of Analysis

Proceeding on the basis of average values, select time stations $t_0 \ldots t_{n-1}$, $t_n \ldots t_f$ at sufficiently close intervals to accurately define the applied blast pressure curve (see figure 2.4.5-40) and resistance function, assuming a straight line variation in resistance and pressure in the time interval.

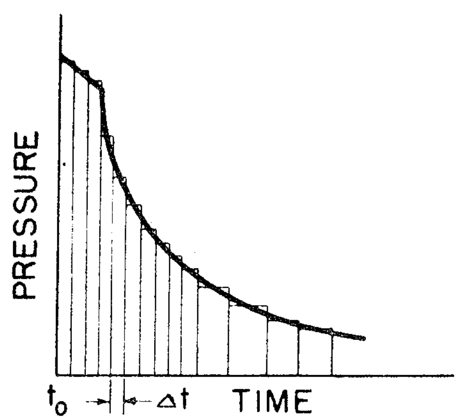


FIG. 2.4.5-40

The table of figure 2.4.5-41 is then set up for computation and the average force, F , exerted by the blast pressure of figure 2.4.5-40 is listed in column 3 for each time interval. The force F is computed as the wall reaction at the top.

---

(30) Dr. N. M. Newmark, "Plastic Limit of Design and Analysis of Building Frames for Impulsive Loads"

1	2	3	4	5	6	7	8	9	10	11	12	13	14
t	Δt	$\overline{F}$	$\overline{R}$	$\overline{F}+\overline{R}$	I	Δv	v_n	$\overline{v}$	Δx	x	P	R_o	$\overline{R}$
t_0							—			—			
	—	—	—	—	—	—		—	—				—
t_1							—			—	—	—	
.													
.													
.													
t_{f-1}							—			—	—	—	
	—	—	—	—	—	—		—	—				—
t_f [1.]							—			—	—	—	
Chosen time Station	Time Increment	Average Translational Load (known)	Assumed Average Resistance during Δt	Net average translational load	Impulse = $(\overline{F}+\overline{R})\,\Delta t$	$\Delta v = \frac{I}{M}$	$v_n = v_{n-1} + \Delta v$	Average velocity = $\frac{v_{n-1}+v_n}{2}$	$\Delta x = \overline{v}\Delta t$	$x_n = x_{n-1} + \Delta x$	Axial Load at t_n	Resistance at t_n (computed)	$R_{avg.} = \frac{R_{n-1}+R_n}{2}$

1. Analysis completed at t_f.

FIG. 2.4.5-41

The frame resistance ($\overline{R}$ of Col. 4) will be equal to the average of the column shears, H , at the beginning and end of each time increment. H , as described previously, is dependent on the deflection, x , and the axial load, P , at each time station. For each time interval the value of R for the preceding station is known, but the value for the end of each time increment must be found by successive approximations. While the first trial value may be obtained by assuming the same R as obtained at the end of the preceding step and then correcting this value in succeeding trials, convergence may be hastened if the first trial value is estimated by mental extrapolation. In this case the convergence is rapid and may be obtained in one or two trial computations.

Having the assumed $\overline{R}$ the subsequent steps are successively:

Col. 5 Find the net force, $\overline{F} + \overline{R}$, accelerating the mass

Col. 6 Record the applied impulse during the time interval $(\overline{F} + \overline{R})\,\Delta t$

Col. 7 Find the change in velocity $\Delta v = \dfrac{(\overline{F} + \overline{R})\,\Delta t}{m}$

Col. 8 Add to find the total velocity at each time period

$$v_n = v_{n-1} + \Delta v$$

Col. 9 Average the preceding and final velocities to obtain the average velocity for the time interval:

$$\overline{v} = \frac{v_{n-1} + v_n}{2}$$

Col. 10 Find the change in displacement during the time interval;

$$\Delta x = \overline{v} \cdot \Delta t$$

Col. 11 Add the change in displacement during the time interval to the preceding total displacement to find the total displacement at the end of the step;

$$x_n = x_{n-1} + \Delta x$$

Col. 12 Find the axial load P for each particular time by adding the dead load and the appropriate roof pressure load.

Then, having the displacement (Col. 11) computed for the assumed resistance and the axial load (Col. 12), and knowing the properties of the column as shown in figure 2.4.5-42, the resistance for the computed displacement may be compared with the trial resistance.

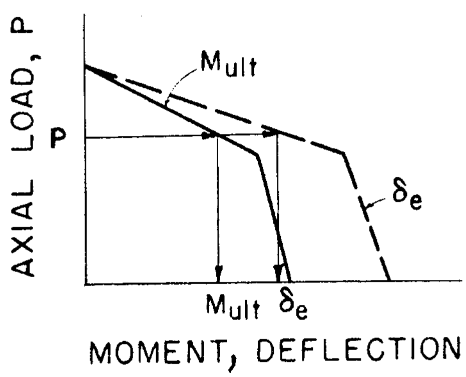


FIG. 2.4.5-42

[redacted]

If the displacement, x_n, is less than the yield point deflection, δ_e, the moment in the column at time t_n will be:

$$M_{act} = \frac{x}{\delta_e} \cdot M_{yp}$$

and

$$R_n = \frac{M_{top} + M_{bott.} - Px}{h}$$

assuming straight line variation of R, the average R for the time Δt_{n-1} to Δt_n will be:

$$R_{av.} = \frac{R_{n-1} + R_n}{2}$$

The computed value of $R_{av.}$ is then compared with the assumed value. If the two values of resistance are not identical a new is assumed for a second trial.

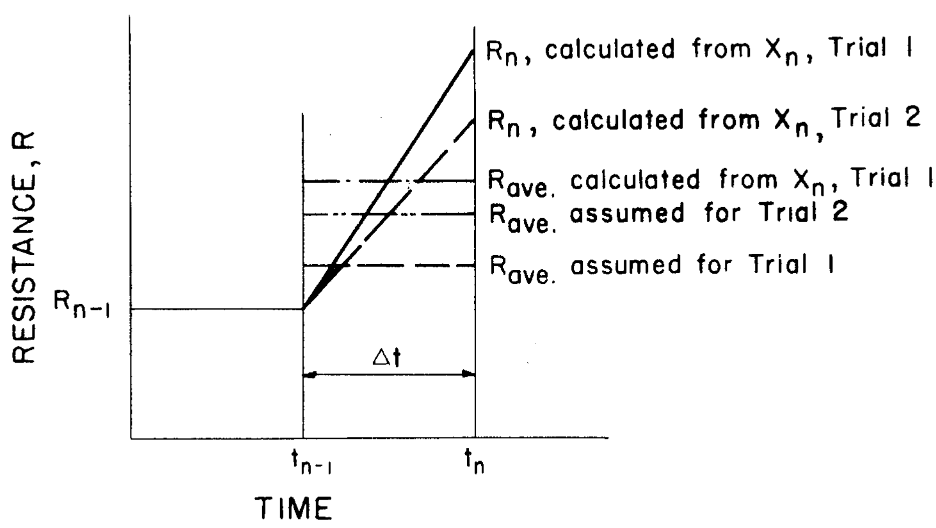


FIG. 2.4.5-43

[redacted]

By this process, $\overline{R}$ may be determined within the desired precision, and the next time increment may be analyzed in a similar manner. The process is repeated until the frame finally comes to rest as indicated by a velocity of zero in Column 8 of the computation table, or until the frame fails, as indicated by deformations exceeding the criteria for failure.

After the deflection proceeds further the columns become plastic, the moments at the column ends will vary with changes in the axial loads. The shear resistance will remain a function of both M and $P \cdot \delta$. The plot of the several typical curves of such an analysis will take the general form shown in figure 2.4.5-44

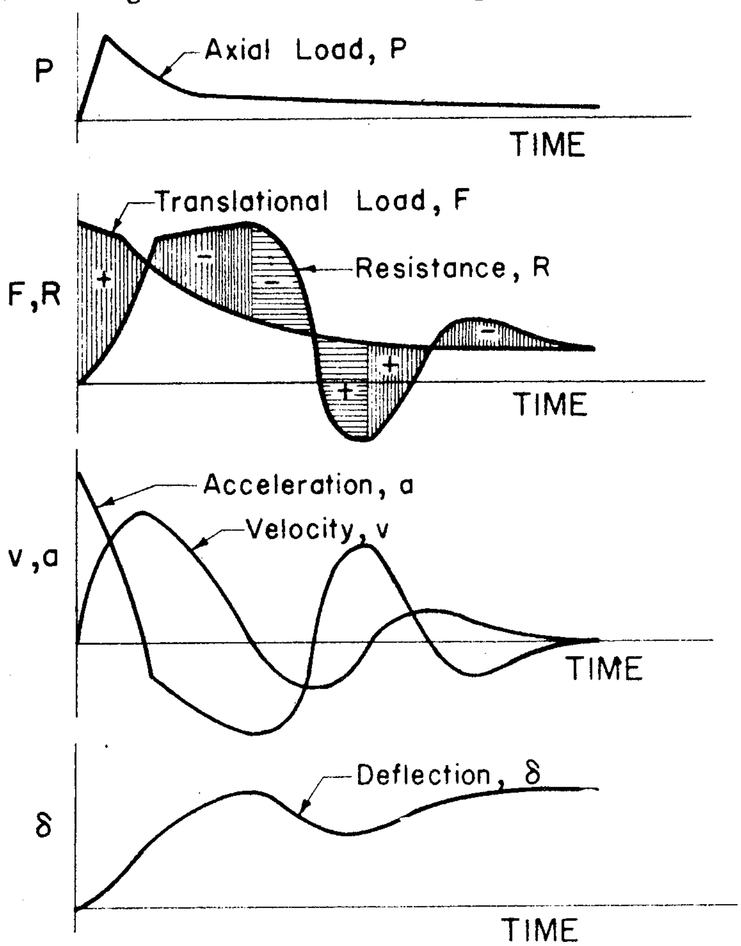


FIG. 2.4.5-44

As mentioned previously, the method presupposes that the elastic strain of the girders and column-girder connections respectively are proportional to the stresses. The error due to neglecting the elastic redistribution of the moments is negligible as the deflection caused

[redacted]

by top elastic end rotation differences in a factor is only a small part of final movement. After the columns develop full plastic resistance the girder rotation remains constant and need no longer be considered.

b. Multi-Story Frame

(1) General Description

As shown in figure 2.4.5-45 a three story frame may be idealized in the same manner as the one story frame discussed above.

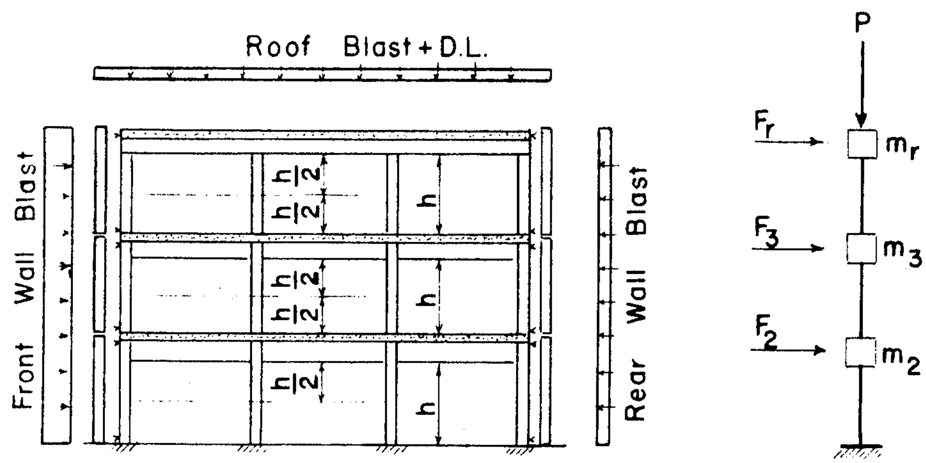


FIG. 2.4.5-45

The mass m for each floor level is assumed to include the weight of the floor slab and supporting beams, the rigidly attached live load, and the weight of walls and columns extending one-half the story height and one-half the clear column height $\frac{h}{2}$ respectively, above and below the floor.

The error involved in the above approximation is usually small, as the columns are only a small part of the total mass. Where the columns compose a substantial part of the total mass, the portion of the column mass to be allotted to each floor may be obtained in the same manner as is described for the walls for the single story frame. The above discussion also applies to the wall mass.

The applied forces F_r , F_3 and F_2 at the various floor levels are each the algebraic sum of the front and rear wall reactions due to the blast pressure.

The column loads, P ,are calculated from the dead weight and the blast pressure at any time, and the girder shears.

As the frame deflects under the blast load, the pattern of this deflection will be generally as shown in figure 2.4.5-46.

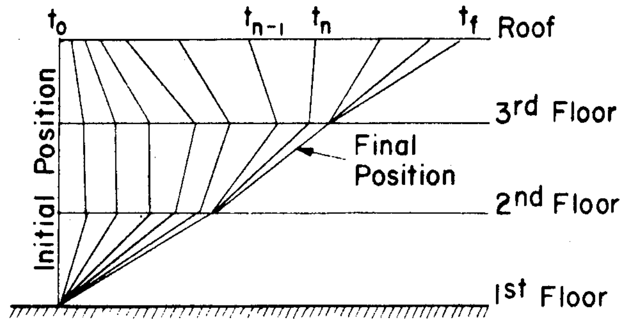


FIG. 2.4.5-46

Because the mass of the roof is approximately equal to the mass of the third floor while the blast load reaction at the roof level is one-half or less of the blast load on the third floor, the acceleration of the roof will be lower than that of the third floor. The roof will then tend to oppose the movement of the third floor during the early stages of motion. This tendency is also helped by the rapid drop in pressure for the top story as shown by the pressure-time curves, Appendix I.

(2) General Design Procedure

The analysis, or rather synthesis of this action proceeds similarly to that of the single story frame in that each story is analyzed as a single unit during each time increment. In the three-story frame, however, the relative motions of the floor masses must be made consistent in each time interval before proceeding to the next time increment. Thus at time t_o , all column shears are 0 , and

$$a_{r_o} = \frac{F_{ro}}{m_r}, \quad a_{3_o} = \frac{F_{3o}}{m_3}, \quad a_{2_o} = \frac{F_{2o}}{m_2}$$

at time t_1

$$a_{r_1} = \frac{F_{r_1} + R_{r_1}}{m_r}, \quad a_{3_1} = \frac{F_{3_1} + R_{3_1}}{m_3}, \quad a_{2_1} = \frac{F_{2_1} + R_{2_1}}{m_2}$$

etc.

At any given time, such as t_2 for example, the displacement and shears for the ideal structure are as shown in figure 2.4.5-47.

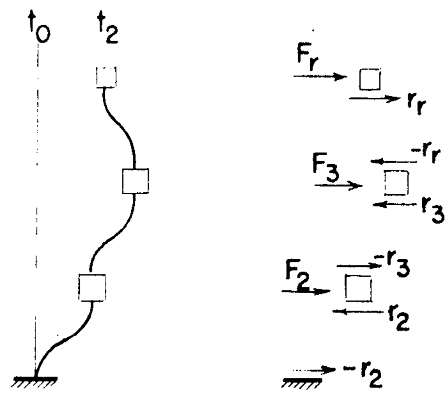


FIG. 2.4.5-47

The analysis of each floor will not differ in method from the analysis of the one-story frame except that there are more elements, and hence the bookkeeping involved in the analysis is more complicated. An outline of the form of computation adopted is shown in figure 2.4.5-48. Note that a separate form is used for each floor, and that a formal sign convention is employed, although this sign convention may be changed to suit the computer. As used here, the columns are designated as to the floor they support, and signs for the resistances of the columns are assigned in accordance with the sense of the shear on the floor supported by the column; thus in figure 2.4.5-47

F_r , F_3 , F_2	are	$+$
r_r	is	$+$
r_3	is	$-$
r_2	is	$-$

[redacted]

The forces acting on the roof mass m_r are F_r & R_r

" " " " " " " m_3 " F_3, r_3 & r_r

" " " " " " " m_2 " F_2, r_2 & r_3

Where R represents the net shear from the columns above and below the particular floors, the method of combining these forces in all computations is as follows:

$R_r = r$ since no other column shears act at this level

$R_3 = r_3 - r_r$

$R_2 = r_2 - r_3$

At time t_2 the sign of the next shears will be

$R_r = r_r = +$

$R_3 = r_3 - r_r = (-) - (+)$

$R_2 = r_2 - r_3 = (-) - (-)$

This system of sign convention was adopted so that only one sign need be assigned to a column shear (resistance), and is explained here so that subsequent numerical computations may be easily followed.

(3) Tabular Arrangement

The tabular form adopted for the three-story frame analysis is shown in the table of figure 2.4.5-48. A similar table is set up for each floor mass.

[redacted]

18	$\bar{R}_{calc.}$					Total Average Calculated Resistance During $\Delta t = \bar{r} - \bar{r}_a$
17	$\bar{r}_a$					Average Resistance Of Columns Supporting Floor Above
16	$\bar{r}$					Average Resistance $= \dfrac{\bar{r}_{n-1} + \bar{r}_n}{2}$
15	r_n					Calculated Resistance Of Columns Supporting This Floor
14	Σ	0	. . .			$\Sigma \Delta x_b - \Delta x$ = Total Deflection Of Floor Below Relative To This Floor
13	$\Delta x_b - \Delta x$					Deflection Of Floor Below Relative To This Floor During Δt
12	Δx_b					Deflection Of Floor Below During Time Interval
11	x	0	. . .			$x_n = x_{n-1} + \Delta x$
10	Δx					$\Delta x = \bar{v} \cdot \Delta t$
9	$\bar{v}$					$\bar{v} = \dfrac{v_{n-1} + v_n}{2}$
8	v_n	0	. . .			$v_n = v_{n-1} + \Delta v$
7	Δv					$\Delta v = \dfrac{I}{m}$
6	I					Impulse $= (\bar{F} + \bar{R}) \Delta t$
5	$\bar{F} + \bar{R}$					Net Translational Force
4	$\bar{R}_{ass'd}$					Assumed Average Resistance For Δt
3	$\bar{F}$					Average Load For Δt
2	Δt					Time Increment
1	t	t_0	t_2 . . .	t_{f-1}	t_f[1]	Chosen Time Stations

1. Analysis completed at t_f.

FIG. 2.4.5-48

(4) Detailed Procedure for Analysis

The analysis may proceed as follows:

Column 1 List the time stations $t_0 \ldots\ldots t_{n-1}, t_n \ldots\ldots t_{final}$

2 Indicate the time intervals being used, $\Delta t = t_n - t_{n-1}$

3 Enter the average blast force $\overline{F}$ for each increment of time

4 Assume an average resistance, $\overline{R}$, for the given time increment.

5 Compute the net force acting on the floor mass as the algebraic sum $\overline{F}+\overline{R}$

6 Compute the impulse transmitted to the floor mass during the time interval as $I = (\overline{F}+\overline{R})\Delta t$

7 The change in velocity during the time interval is the impulse divided by the mass or $\Delta v = \frac{I}{m}$.

8 Compute the velocity of the floor mass at the end of the time interval as $v_n = v_{n-1} + \Delta v$

9 Compute the average velocity, $\overline{v}$, during the time interval as $\frac{v_{n-1}+v_n}{2}$

10 Δx, the displacement during the given time interval $= \overline{v}\Delta t$

11 and the total displacement at the end of the given time interval $x_n = x_{n-1} + \Delta x$

This much of the operation is completed individually for each floor. The following steps are required to make the displacements of the various stories compatible with each other. Then during this time (see figure 2.4.5-49)

12 Let Δx, figure 2.4.5-49, represent the displacement of any given floor in the time interval being considered and let Δx_b represent the corresponding displacement of the floor below. Deflection in the direction of the blast will be taken as positive.

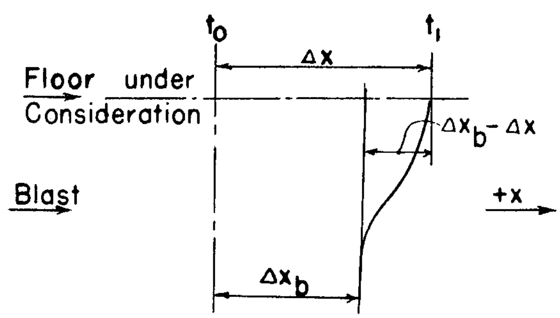


FIG. 2.4.5-49

[redacted]

13 & 14 The change in relative displacement of the two floor masses (causing column shear) $\Delta x_b - \Delta x$ is computed in column 13. The sign of the difference, $\Delta x_b - \Delta x$ and of $\Sigma(\Delta x_b - \Delta x)$, column 14, automatically assigns the proper sign to the resistance r for the floor under consideration.

15 Once the Σ of relative deflections of the columns are obtained, r_2, r_3 and r_r can be determined for any time, t_n, as described previously.

16 Compute the average resistance of the columns below as

$$\bar{r} = \frac{r_{n-1} + r_n}{2}$$

17 $\bar{r}_a$ is the average resistance of the columns above with sign unchanged.

The net average resisting force on this floor during the time interval is the algebraic difference of the average resistances of the columns above and below the floor or

$$R_{calc.} = \bar{r} - \bar{r}_a$$

When this operation is completed for all three floors, the calculated resistances are compared with those assumed for the first trial. If the assumed values differ from those given by the resistance function for the computed deflections, new resistances are selected and the operation repeated as many times as is necessary to produce satisfactory agreement between the assumed and the calculated resistance for each floor. When this is accomplished, the analysis is complete for that time interval, and the computation proceeds to the next interval. This process continues until the frame comes to rest.

This particular frame analysis can be performed in from 12 to 20 hours, once preliminary computations are completed, and can be regarded as a practical analytical procedure. The time required is not greatly in excess of that required for a complete elastic analysis of a similar three-story, three-bay frame.

(5) <u>Effect of Details of Construction</u>

The analysis for the three-story frame, as well as the analysis of one-story frames, is influenced very much by the following details of construction of the building:

(a) <u>Wall panels</u>

As these walls must be made discontinuous at each floor level to eliminate their participation in the lateral resistance of the frames, the blast loads transmitted to the frame will be simple beam reactions.

[redacted]

As discussed in Section 2.3.5, the period of vibration of low mass wall panels acting in the elastic range is so short relative to the period of the frames themselves that the effective impulse delivered to the frame is approximately the same in intensity and duration as the impulse on the panel. If the panel is designed to act in the plastic range, its reaction curve will be of the form shown in figure 2.4.5-50 which may again be closely approximated by use of the blast pressure curve.

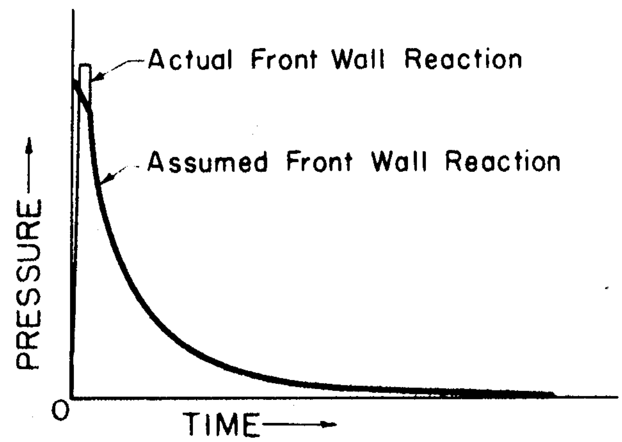


FIG. 2.4.5-50

Since the reactions of the non-participating wall panels are not complicated by continuity of the panels, the front and rear wall panel loads may be combined algebraically and hence the load to the frame is of the same general shape as, and can be plotted or tabulated directly from the translational force curves, discussed in Section 2. The curves generally are of the shape shown in figure 2.4.5-51.

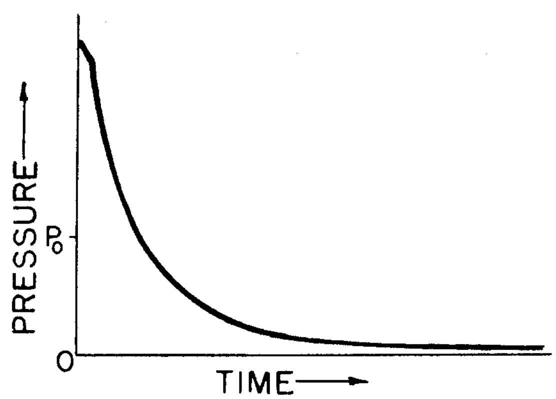


FIG. 2.4.5-51

[redacted]

This panel behavior is typical of the action of the steel wall panels. The effect of slow-moving, high-mass wall panels will be described in Section 2.4.5 C(8).

(b) Roof Panels

The roof panels are all relatively massive and should be designed to resist the loads plastically. Their load, resistance, and reaction curves are generally of the form shown in figure 2.4.5-52.

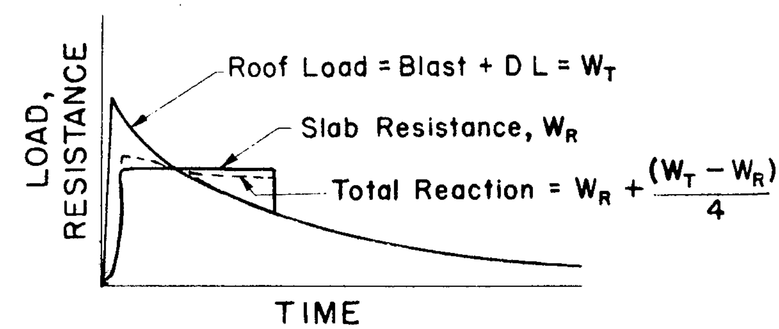


FIG. 2.4.5-52

Since the load transmitted to the columns is a function of the roof slab resistance, it is possible to control the magnitude of column loads to some extent. By using relatively heavy roof slabs, with small percentages of steel instead of lighter panels which require more steel, the rate of slab deformation is lowered and thus a smaller plastic resistance will be needed. This artifice results in lower peak axial loads acting for a somewhat longer time.

The roof girders, receiving the roof panel reactions, are designed to remain within the elastic range. Because of their short natural period of vibration the reactions to the columns may be considered as similar to the slab reactions.

[redacted]

(c) Effect of Traveling Roof Load on Columns

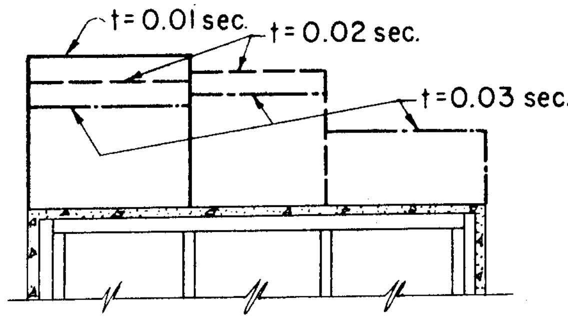


FIG. 2.4.5-53

Figure 2.4.5-53 illustrates the variation of roof loads with time. It is apparent that this variation of load on the elastic roof girder could induce large and rapidly changing moments and shears in the columns which are independent of the lateral deflection of the frame. Since an elapsed time of approximately 0.03 seconds is required for the pressure wave to reach the rear edge and as this represents a considerable increment of time in the frame analysis, it seemed possible that an appreciable error might be introduced if this effect were not accounted for. An investigation made for the purpose of evaluating the magnitude of the error involved indicated that its effect on the analysis was negligible.

However, for the design of the frames described in later sections, the variable roof load was included for computation of column loads because the final pressure curves indicated a considerable variation in the shape, intensity, and duration of the pressure curve at various points along the roof.

As noted in the discussion of single-story frames, comparative studies were made to determine the error introduced by the assumption that the girders are infinitely stiff as compared to the columns. This comparison cannot be generalized to cover all cases, but where the ratio of column stiffness to the girder stiffness is similar to that in the test structures, this assumption has no significant effect on the accuracy of the analysis.

[redacted]

However, the importance of a rapid development of the column resistance can best be illustrated by the consideration of a single-story frame as follows:

The characteristic velocity curve for the roof of a single-story frame is as shown in figure 2.4.5-54.

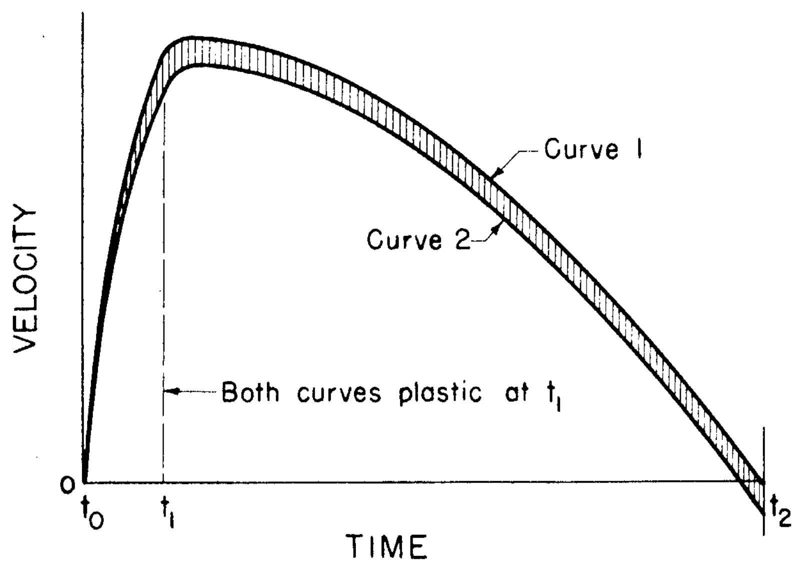


FIG. 2.4.5-54

Curve 1 of figure 2.4.5-54 shows the velocity of the roof mass of a frame having any given resistance to lateral displacement.

Curve 2 illustrates the velocity curve of the same roof mass under the same applied load and with the same final frame resistance except that the resistance of the frame of curve 2 is developed earlier than for the frame of curve 1. The difference in total displacement is represented by the shaded area between the two curves.

It is apparent that the most effective resistance is the one applied the soonest and to accomplish this the girders and column-girder connections should be as stiff as practicable.

(6) <u>Summary of the Detailed Multi-Story Design Procedure</u>

(a) <u>Walls:</u> In order to obtain the lateral and vertical loads on the frames, the wall and roof panels are designed prior to design of the frames; the panel action being quite independent of the frame action. The design procedure for these members is outlined in Section 2.5.3.

(b) <u>Roof Girders:</u> The preliminary size of the roof girders can be obtained without regard to column moments, since the relative

frame and roof loading are such that the moments in the girders due to roof load almost completely control their design. In the design the girder size may be tentatively established on the basis of the roof load only. This girder is then checked for adequacy against the added loads obtained from the frame analysis.

(c) Floor Girders: In frames similar to those in Sections 2,3,5 and 6 of the test structure, the design of the floor girders is almost completely controlled by the column moments. Preliminary sizes are assumed on the basis of predicted column moments; these sizes are later revised as required by actual column moments. This revision usually will not affect the frame analysis since the column moments may be foreseen fairly accurately. It should be noted that, for riveted construction, the details of the column to girder splice may control both column and girder size, will surely influence both, and should be carefully investigated before the frame analysis is begun.

(d) Columns: The basis for the preliminary selection of column sizes is discussed in detail in Section 2.4.5-D. The suitability of these choices must then be checked by a step-by-step frame analysis in item (f).

(e) Preliminary Computations for Frame Analysis:

(1) Prepare P-M and P-δ curves for columns

(2) Tabulate wall panel reactions

(3) Tabulate the column loads

(4) Compute story masses

(f) Frame Analysis: Proceed with the frame analysis as described in Section 2.5.3-B.

4. Method of Design Analysis (Walls Participating)

a Resistance Functions - Frames

The addition of continuous walls which are connected to and are either restrained or unrestrained by the floor slabs and which help thereby to resist the horizontal displacement of the frame, in no way change the resistance function of the frame proper. The earlier discussion of column resistances and frame action is not modified by the action of the wall, and the frame analysis remains fundamentally the same. However, the analysis of the action and effect of the wall itself necessitates modification of the design procedure.

b Resistance Function of the Wall

Walls of the type used in the test structure are supported by hinge-type joints at the intermediate floor levels, are restrained

[redacted]

at the lower end and are fixed to some degree at the top, depending on the relative rigidity of the wall and roof systems. It is convenient to consider their action separately from the action of the frame. The reaction at each support is the frame load. These reactions, due to blast pressure and to relative deflection of the floors, are computed on the assumption of continuity in each time interval.

A deviation from this type of multi-story construction may be made by restraining the wall at each floor level with spandrel beams or floor slabs as shown in figure 2.4.5-55. The walls and columns can then be treated as integral parts of the resistance function.

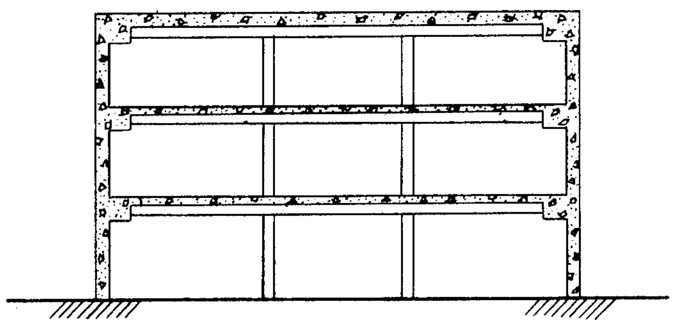


FIG. 2.4.5-55

Exterior columns were used in the test structures and thus the walls support no appreciable vertical live load. When the walls are fixed at each floor level, exterior columns may be omitted and the walls must then support a portion of the roof load.

c Description of Participating Wall Behavior

(1) General Behavior of the Wall of a Single Story Frame

Since the single story frame with participating walls is the simplest in behavior, this type will be described first.

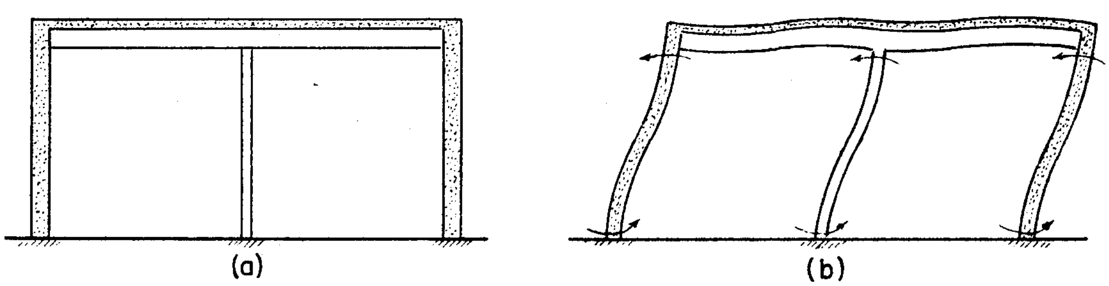


FIG. 2.4.5-56

[redacted]

It is obviously advantageous to have the wall panels restrained at top and bottom, rather than simply supported, because of the gain in resistance to horizontal blast load. Furthermore for some types of construction materials, such as reinforced concrete, this restraint may be obtained with practically no added cost. Figure 2.4.5-56 illustrates that, apart from the additional effect of the local blast load, the walls of a single-story frame will act in exactly the same manner as columns if the top end is restrained by the roof slab when the frame deflects horizontally. The wall thickness should be approximately 1 foot as a matter of practical construction. Since the columns themselves may only be from 12 to 20 inches square, the resistance of the wall to horizontal deflection will usually exceed the combined resistances of the columns even if the walls develop resisting moments at the bottom only. In view of this added strength, the economy of allowing the walls to participate is obvious. The design, analysis and construction of the single-story frame are but little, if at all, complicated by this action.

(2) <u>General Behavior of Multi-Story Walls</u>

In the case of multi-story frames, the action of the participating walls, apart from their function of resisting the blast loads by flexure between floor levels, is somewhat more complex. The following description is concerned with continuous walls which are fixed at the upper and lower ends and rest on yielding supports at each floor level.

Diagrammatically the wall is shown in figure 2.4.5-57.

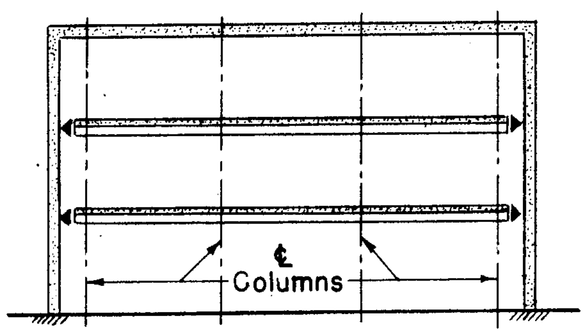


FIG. 2.4.5-57

The frame deflection due to horizontal blast pressure will still follow the sequence shown in figure 2.4.5-46 and the action of the wall, exclusive of blast load on it, will be that of a continuous beam on yielding supports. The wall will be forced into the

[redacted]

configuration of the frame producing reactions on the frame due to deflection only which are similar to the forces shown in figure 2.4.5-58.

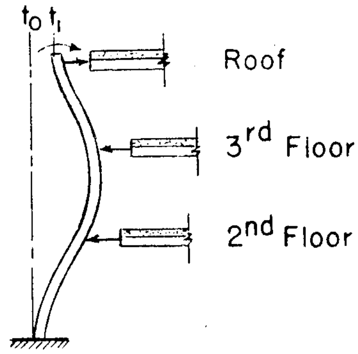


FIG. 2.4.5-58

So long as the wall is elastic, these reactions may be computed if the relative deflections of the various floors are known by treating the wall as a continuous beam. The relative deflections of the floors are determined in the frame analysis and are also used for the computation of column shears. It is evident that the frame deflections determined for each time interval are a function of the wall reactions as well as the column shears.

The wall will eventually undergo such large deformations that plastic moments will develop at various points. The analysis of the wall must be compatible with such plastic action.

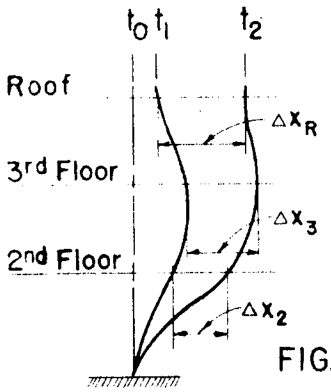
(3) Detailed Behavior of the Rear Wall

The process of analyzing the rear wall is done in conjunction with the frame analysis since the resistance of both the frame and wall are functions of the same deflection and are combined to give the total resistance.

At time t_0 all wall reactions will be zero. During the first time interval, $\Delta t_{0 \to 1}$, the wall will move into the position shown in figure 2.4.5-58. At time t_1 the relative deflections of the supports are taken from the frame analysis, and, knowing the elastic stiffness of the wall, the moments in the wall are determined. These moments plus the blast load will determine the wall reactions at time t_1 . These are then tabulated, and considered as part of the total load transmitted to the frame.

In the Second time interval, $\Delta t_{1 \to 2}$, the floors move through an additional displacement, Δx , and the changes in relative deflections $\Delta \delta$ of the floors during this time interval are then determined as shown in figure 2.4.5-59.

[redacted]



$$\Delta \delta_R = \Sigma(\Delta x_3 - \Delta x_R)$$

$$\Delta \delta_3 = \Sigma(\Delta x_2 - \Delta x_3)$$

$$\Delta \delta_2 = -(\Delta x_2)$$

FIG. 2.4.5-59

These changes in relative deflections for the time, Δt_{1-2}, are used again to determine fixed-end moments for the walls which are then distributed and tabulated as the change in moment, ΔM, occurring during the time increment from t_1, to t_2. The quantities, ΔM, developed during Δt_{1-2} at each support are then added to the moment, M_1, determined in the preceding step for time t_1, thus providing the total moment M_2 at the time t_2.

During some time interval, the blast pressure begins to act on the rear wall. To include the effect of this local pressure on the wall, the difference in wall loads at the beginning and end of the interval, are computed for each panel, and fixed-end moments are determined for these loads. The fixed-end moments due to the local wall load, and the relative deflection during the time interval, are combined and the total change in fixed-end moment is distributed to give the actual change in moment at each support. The necessity for working with increments of load and relative displacement is apparent when the wall begins to yield at any of the supports.

If the wall is entirely elastic at the time, t_n, but the yield point is exceeded at a support at the time, t_{n+1}, the above procedure must be modified. The change in moments for the time interval are determined for the elastic condition and the moments M_{n+1}, are compared with the yield point moment at each respective support. At any support which has yielded, the part of the moment which is in excess of the yield point moment is redistributed. That is, after the wall reaches its yield value at any point, it is considered hinged at this point for additional moments of the same sign, but is considered continuous and elastic under the application of moments of the opposite sign. This procedure is based on the assumption that moments will reverse as shown in figure 2.4.5-60.

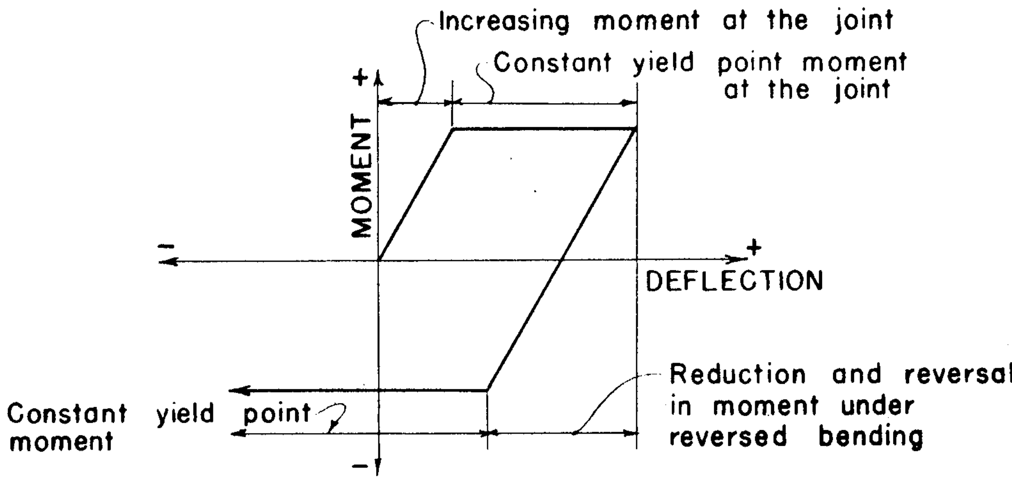


FIG. 2.4.5-60

(4) <u>Effect of Plastic Hinges Between Supports</u>

It should be noted that during the frame analysis the possibility of plastic hinges between supports was neglected. This condition may be analyzed after and separately from the frame analysis. The reasons for this procedure are that (1) yielding between supports occurs only under special conditions of loading and usually lasts for extremely short durations of time and (2) this phenomenon affects the motion of the wall for only a very short interval of time.

Despite the rapid rates of loading experienced by the walls, a corresponding increase in strength was neglected because no definite information is available concerning the effects of moment reversal after large plastic strains such as are developed by the wall in local bending under the initial blast loads. For this reason, and in view of statements (1) and (2) above, it is felt that errors introduced by assuming the walls yield only at the supports have no appreciable effect on the degree of accuracy of the analysis.

(5) <u>Effect of Simultaneous Plastic Yield at More than One Support</u>

If yielding occurs at both intermediate supports in the same time interval, it is necessary, for practical reasons, to use an additional approximation concerning wall action. The excess moment at the support which has yielded the most is distributed first. Then, if a moment greater than the elastic yield point-moment still exists at another support, the excess is redistributed, assuming that the support which yielded first is hinged, regardless of the sign of the moment carried to it. It is apparent that the moments at the supports in question must be very near the yield point values

[redacted]

and therefore no great error is made in the final moments by this procedure. The problem may be avoided by decreasing the time interval so that the yield point moment is reached at only one support during any time interval.

(6) Detailed Behavior of the Front Wall

In the case of the front wall, which is plastic almost instantly because it resists the horizontal blast load, the procedure for wall analysis is similar to that for the rear wall except for the initial effects of local bending. In accordance with the wall resistance function shown in figure 2.4.5-61, it is assumed that plastic resistance for all three wall panels stops at the same time, this time being the average value of t_f for the three panels. The wall reactions $W_R + \frac{W_B - W_R}{4}$ up to the average time, t_f , are proportioned with regard to the value of the plastic moments at each support.

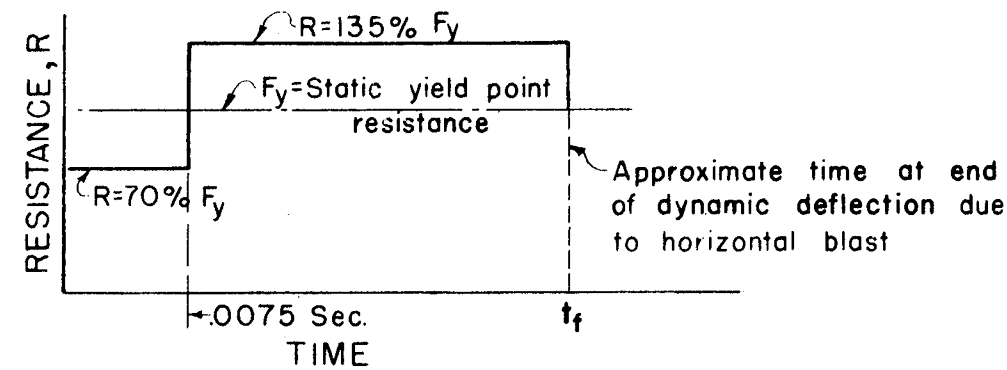


FIG. 2.4.5-61

It was found from the panel and frame analyses that the blast pressure panel rotations at the supports are large compared to the rotations caused by frame deflection. Thus the frame deflection has little effect on the wall panel moments during this time. After the assumed time at which the panel stops moving, the values of the moments at the supports are reduced from their dynamic to their static strength values. When the dynamic plastic resistance of the front wall ceases, this member is treated in a manner similar to the rear wall.

The seemingly arbitrary change from increased strength under dynamic loading for the local blast loads to static load strength for the later loads seems to be the best procedure that can be justified by the limited quantity of stress-strain data which is available for materials under dynamic loads.

[redacted]

20	$(\bar{F}+\bar{R})_{calc}$	| |	Calculated Average Net Translational Force
19	$\bar{F}_{calc}$	| |	Calculated Average Load During Δt = Average Of Σ Wall Reactions At t_{n-1} & t_n
18	$\bar{R}_{calc}$	| |	Total Average Calculated Resistance During Δt $= \bar{r} - \bar{r}_a$
17	$\bar{r}_a$	| |	Average Resistance Of Columns Supporting Floor Above
16	$\bar{r}$	| |	Average Resistance $= \dfrac{\bar{r}_{n-1} + \bar{r}_n}{2}$
15	r_n	0 | . . . | |	Calculated Resistance Of Columns Supporting This Floor
14	Σ	0 | . . . | |	$\Sigma\,\Delta x_b - \Delta x$ = Total Deflection Of Floor Below Relative To This Floor
13	$\Delta x_b - \Delta x$	| |	Deflection Of Floor Below Relative To This Floor During Δt
12	Δx_b	| |	Deflection Of Floor Below During Δt
11	x	0 | . . . | |	$x_n = x_{n-1} + \Delta x$
10	Δx	| |	$\Delta x = \bar{v} \cdot \Delta t$
9	$\bar{v}$	| |	$\bar{v} = \dfrac{v_{n-1} + v_n}{2}$
8	v_n	0 | . . . | |	$v_n = v_{n-1} + \Delta v$
7	Δv	| |	$\Delta v = \dfrac{I}{m}$
6	I	| |	Impulse $= (\bar{F} + \bar{R})\,\Delta t$
5	$(\bar{F}+\bar{R})_{ass'd}$	| |	Assumed Net Translational Force
4	$\bar{R}_{ass'd}$	| |	Assumed Average Resistance For Δt
3	$\bar{F}_{ass'd}$	| |	Assumed Average Load For Δt
2	Δt	| |	Time Increment
1	t	t t_1 . . . t_{f-1} t_f	Chosen Time Stations

1. Analysis complete at t_f

FIG. 2.4.5-62

[redacted]

5. General Method of Analysis

The general form of computation for the analysis of frames with participating walls is shown in figure 2.4.5-62. Up to columns 19 and 20 the procedure is the same as outlined for the frame with non-participating walls (Section 2.4.5-C3).

Upon determination of the frame deflection due to assumed frame loads it is necessary to compute the actual average wall reaction at each floor level, $\overline{F}$, as well as column resistance in each story, $\overline{R}$. The wall reactions are now functions of frame deflection as well as blast pressure. For each floor level the average value of the wall reaction is entered in column 19. The actual average net translational force, $\overline{F}+\overline{R}$ which is entered in column 20 for each floor level is then compared with the assumed values in column 5. If these values do not show adequate agreement this process is repeated. When the assumed and calculated values agree within the desired precision, the analysis may proceed to the next time interval.

For time intervals of the magnitude, $\Delta t \leq 0.025$ seconds, the convergence between assumed and calculated values of $\overline{F}+\overline{R}$ for loads and frames similar to those of the test structure is fairly rapid for frames with non-participating walls. An experienced computer can estimate the first trial R quite accurately, and by judicious interpolation between the calculated and the assumed R hasten satisfactory agreement. Barring mechanical errors no more than two or three trials are required for each step.

In the case of the frames with participating walls, the choice of the trial values of $\overline{F}+\overline{R}$ is more difficult because both the frame loads and column resistances are functions of deflection. If the time intervals are sufficiently small, the assumed and calculated values of $\overline{F}+\overline{R}$ will always converge for time intervals approaching 0.025 seconds in magnitude. The procedure described above may result in a divergence of the assumed and calculated values of $\overline{F}+\overline{R}$, if a poor choice is made for the trial value. As in the case of frames with non-participating walls a computer with some experience will usually arrive at satisfactory results on the third trial.

The preliminary design proceeds similarly to that for frames with non-participating walls. The differences are discussed below.

a Wall Panels

The participating-type wall is most easily achieved by the use of reinforced concrete wall panels which should be designed to resist local blast pressure independently of the frame. For this purpose it is desirable to take advantage of continuity by providing negative reinforcement over the supports. This reinforcement will provide moment resistance to deflection during subsequent frame displacement. However, additional wall strength over that required to resist the local blast pressures is un-

[redacted]

desirable because the increased resistance against frame deflection can be developed more economically in the columns.

Because of their greater mass, the accelerations and velocities are lower and the duration of the plastic action is longer than for steel panels. The reactions of the high mass front wall panels, which have full plastic deformation are shown in figure 2.4.5-63

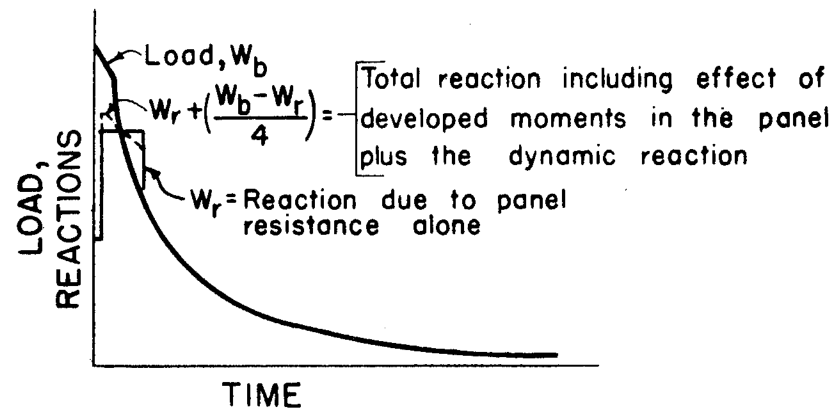


FIG. 2.4.5-63

This type of reaction curve is somewhat less severe than the low-mass panel reaction because its distribution with respect to time is more favorable. This effect does not occur at the rear wall, where it is assumed that the smaller rear wall pressures are resisted in the elastic range and are transmitted without distortion to the frame, except for modifications due to the continuity of the panels. The net translational force during plastic action is shown in figure 2.4.5-64.

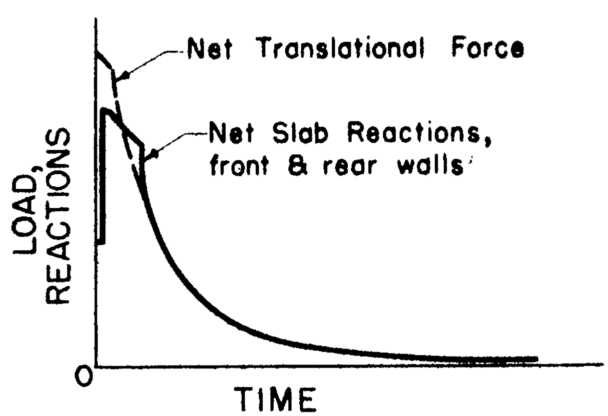


FIG. 2.4.5-64

[redacted]

These characteristics plus the effect of the inertial mass added to the frame itself, result in lower frame accelerations and velocities.

b. Preliminary Column Sizes

The total resistance required between the various floors and the probable contribution of the walls to the total resistance may be determined as outlined in Section 2.4.5-D. The balance of the required resistance must then be supplied by column shears

D. Frame Design: Comparison of Methods of Design and Proportioning

It is difficult to determine the effect of each variable on the action of the 3-story frame. However, a considerable amount of work on preliminary designs was done using simplified assumptions and methods, which permitted the changing of one variable at a time. All of these studies which are listed below were for frames similar to those used in the test structure.

1. Effect of Assumed Shape of the Resistance Curve

Several analyses were made using different assumptions regarding the manner in which the resistance varies in the elastic range. The resistance in the plastic range was assumed constant in all these cases.

Case 1. (a) The acceleration was assumed to be constant in each time interval and a resistance function, shown in figure 2.4.5-65, which increases linearly with strain up to the yield point was used.

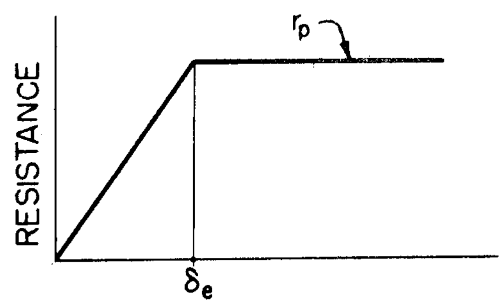


FIG.2.4.5-65

(b) A linear variation in acceleration was assumed in each time interval with the same resistance function as in Case 1 a. This analysis was made by J.T. Penzien and H. A. Williams at Massachusetts Institute of Technology.[31]

(31) J. B. Wilbur and C. H. Norris, "Some Comments Regarding Plastic Limit Design of Building Frames for Impulsive Loads."

A comparison of the methods illustrated by cases 1a and 1b showed identical results within slide rule accuracy and the use of either method does not seem likely to affect the accuracy of the solution.

Case 2. The resistance was approximated by using zero, 50%, or 100% of the full plastic resistance, whichever value came closest to the true resistance. This assumption is illustrated in figure 2.4.5-66.

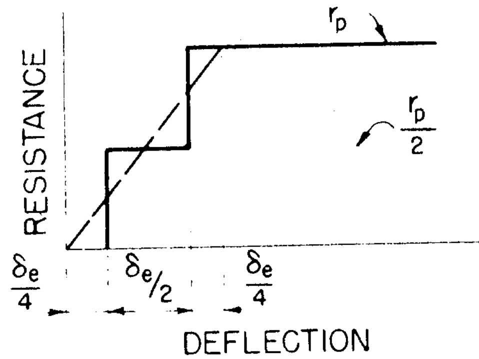


FIG. 2.4.5-66

Case 3. In case 3 it was assumed that the second floor column resistance was equal to one-half the full plastic resistance and that the third floor and roof column resistances were equal to zero during the first time increment. Thereafter, zero or 100% resistance, whichever value was closest to the true condition, was used. Refer to figure 2.4.5-67.

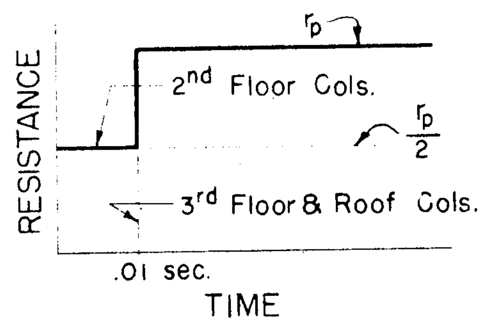


FIG. 2.4.5-67

Case 4. In accordance with deflection anticipated for the end of the first time interval as shown in figure 2.4.5-68, the column shears were assumed to be zero for the third floor and a maximum plastic value of the proper sense for the other floors.

[redacted]

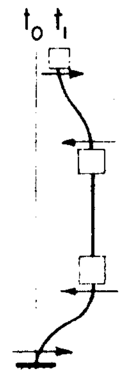
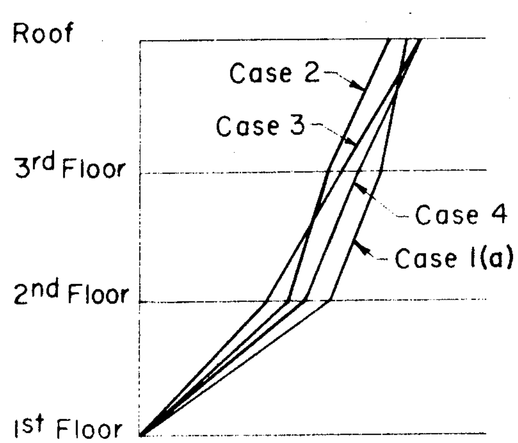


FIG. 2.4.5-68

For all subsequent time increments the zero or 100% values of resistance, whichever came closest to the true value, was used.

The accuracy of the numerical procedure of Case 1 is dependent on the size of the time increment. Therefore, by using a reasonably small time increment for Case 1 and the same time increments for Cases 2 to 4, a comparison of the accuracy of the latter cases can be drawn. The results for Cases 1a,2,3, and 4 are summarized in figures 2.4.5-69 and 70.



TOTAL FLOOR DEFLECTION

FIG. 2.4.5-69

[redacted]

Tabular Comparison of Deflections for Various Cases 1 to 4						
Case	2nd Floor		3rd Floor		Roof	
	δ_{max}	Accuracy	δ_{max}	Accuracy	δ_{max}	Accuracy
1(a)	.44	1.00	.56	1.00	.61	1.00
2	.35	.80	.44	.79	.58	.79
3	.29	.66	.47	.84	.65	1.07
4	.37	.84	.50	.89	.65	1.07

FIG. 2.4.5 — 70

Case 5. In addition to the study referred to in Case 1b, Penzien and Williams (31) analyzed the same frame using the full plastic resistance for all frame deflections. This assumption is illustrated in figure 2.4.5-71.

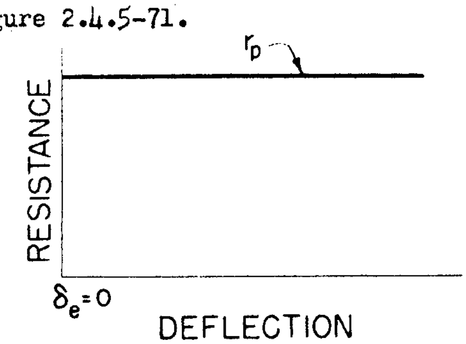


FIG. 2.4.5-71

The comparative deflections obtained by Cases 1b and 5 are shown in figure 2.4.5-72

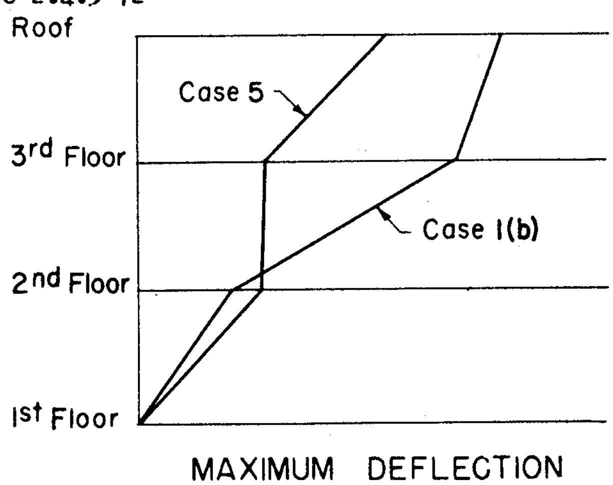


FIG. 2.4.5-72

(31) J. B. Wilbur and C. H. Norris, "Some Comments Regarding Plastic Limit Design of Building Frames for Impulsive Loads"

[redacted]

The wide discrepancy between the results obtained in Cases 1b and 5 compared to the narrow range of variation shown by Cases 1a,2,3 and 4 is to be expected because the assumptions of Case 5 diverge farthest from the physical facts.

Figures 2.4.5-73 and 74 indicate diagrammatically the errors involved in using Cases 4 and 5.

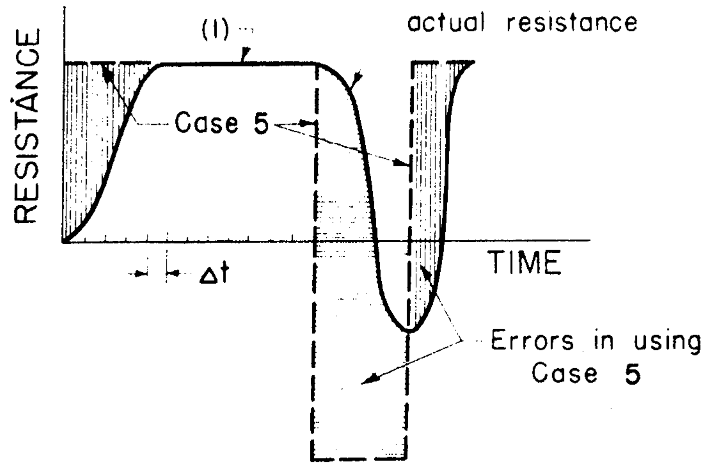


FIG. 2.4.5-73

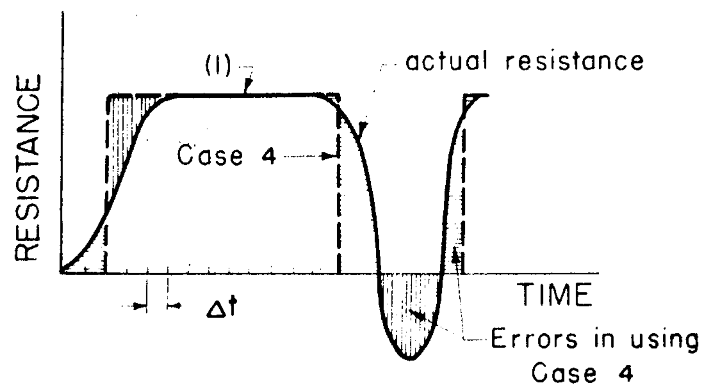


FIG. 2.4.5-74

Case 6. One other method of analysis using a resistance curve as shown in Figure 2.4.5-75 was compared with an analysis based on the method outlined for Case 1(a). In this method the resistance was assigned by inspection in steps of zero %, 25%, 50%, 75% or 100% of plas-

[redacted]

tic resistance, whichever value was closest to the true resistance. The results are shown in figure 2.4.5-76.

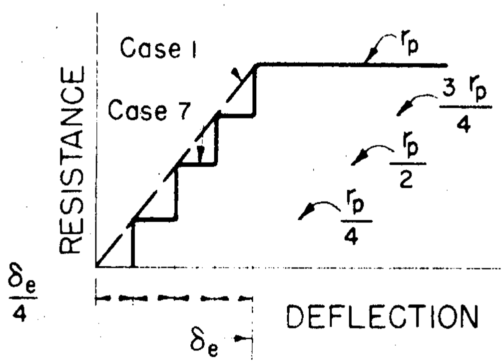


FIG. 2.4.5 - 75

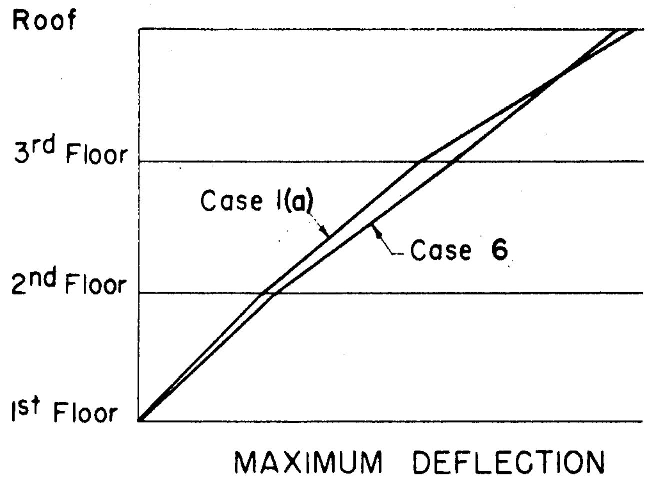


FIG. 2.4.5-76

The method outlined for Case 1a was used in all final analyses, with provision made for the variation of plastic resistance due to the various factors outlined in Section 2.4.5-C.

2. Effect of the Length of the Time Increment

In all of the above methods except Case 1b constant resistance is assumed during each time increment and thus the time of application of a small part of the resistance impulse is shifted within each increment of time. Assuming that the linear rise in resistance is correct, it follows that for longer time increments the error becomes greater.

Comparative multi-story analyses indicated that the error resulting from changing the time increment from 0.005 to 0.010 seconds is insignificant and changing the time interval from 0.005 to 0.020 seconds results in errors in peak velocities and maximum deflections of less than 5%.

Though the above comparison indicates that it is possible to use larger time increments without introducing material errors, the ease in computation is not improved significantly because the convergence of the assumed and calculated resistances is slower. In general little time will be saved by using time increments greater than 0.01 seconds in the elastic range and 0.025 seconds thereafter.

The slight error introduced by assuming a constant plastic resistance at the transition from the elastic to the plastic range, as indicated in figure 2.4.5-77b may be partially offset by assuming that the resistance becomes plastic at the end rather than during the time increment as shown in figure 2.4.5-77a.

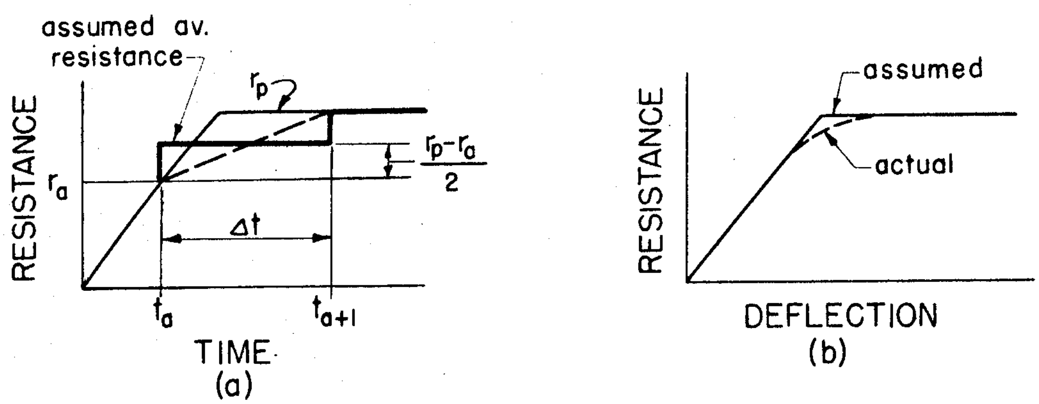


FIG. 2.4.5 -77

3. Effect of Rate of Increase of Resistance

Two studies were made of identical frames to determine the effect of frame stiffness. In case 1 it was assumed that the columns were fixed at the top and bottom. In case 2 the elastic rotation at these points was approximated as shown in Section 2.4.5 C-1. The results of these studies are shown in figure 2.4.5-78.

	Joint Rotation at M_{ult} (radians)			Elastic Deflection δ_e (feet)			Total Deflection (feet)		
Floor	2nd	3rd	Roof	2nd	3rd	Roof	2nd	3rd	Roof
Case 1	0	0	0	.052	.076	.054	.40	.61	.73
Case 2	.0056	.0047	.0022	.086	.135	.093	.38	.72	.93
% Variation				+65	+79	+72	−5	+15	+21

FIG. 2.4.5-78

It is important to note that the results of this comparison as well as those in Section 2.4.5 D-1 imply that most assumptions concerning the resistance functions in the elastic range do not result in serious variations in the total deflection. This is in direct contrast to the more serious effects of errors in estimating the blast load or the plastic resistance. This conclusion would not be true however, if the total deformations were limited to smaller values.

[redacted]

4. Effect of Axial Load

In the preliminary frame analyses it was assumed that the resistance was constant for strains in the plastic range, the variation in bending resistance due to axial load and to load eccentricity being neglected. Figures 2.4.5-79 and 80 show the variation of the computed resistance with time for the steel frame and concrete frame buildings as taken from the final analyses. These analyses were made to determine the effect of blast pressures equal to 135% of the theoretical blast pressure, and included the effects of the axial load and the eccentricity of the axial loads due to relative floor deflections. Because of the lower total resistances of the columns in the concrete frame, and because of the characteristic shape of their P-M curves (see section 2.4.5-C) they are more sensitive to these effects than the steel columns. This is illustrated in figures 2.4.5-79 and 80.

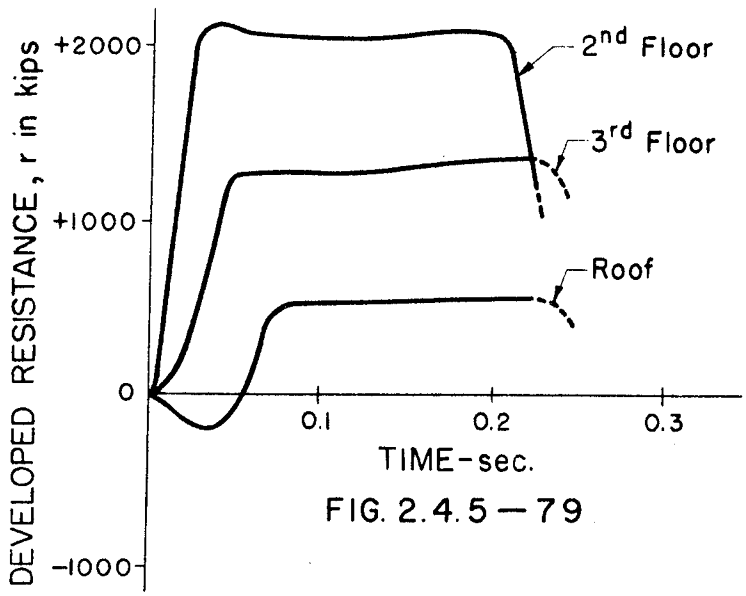


FIG. 2.4.5—79

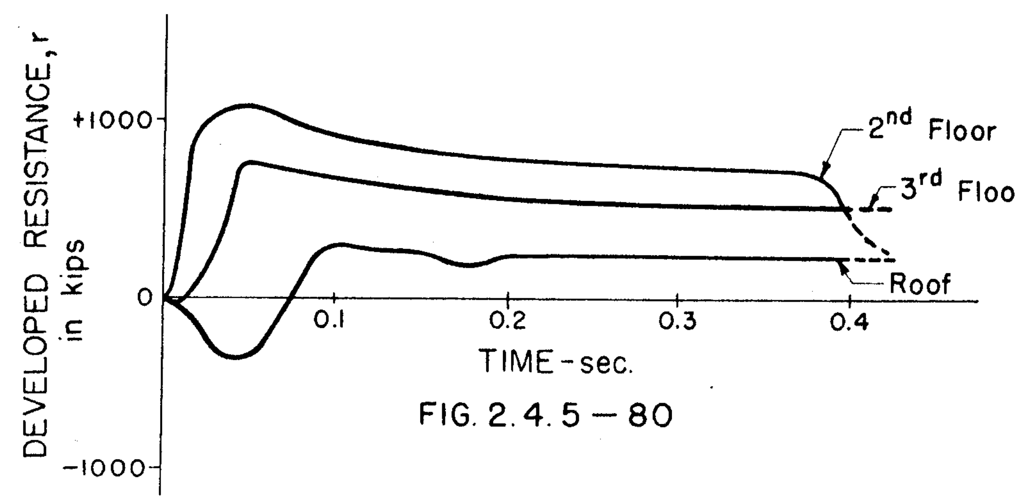
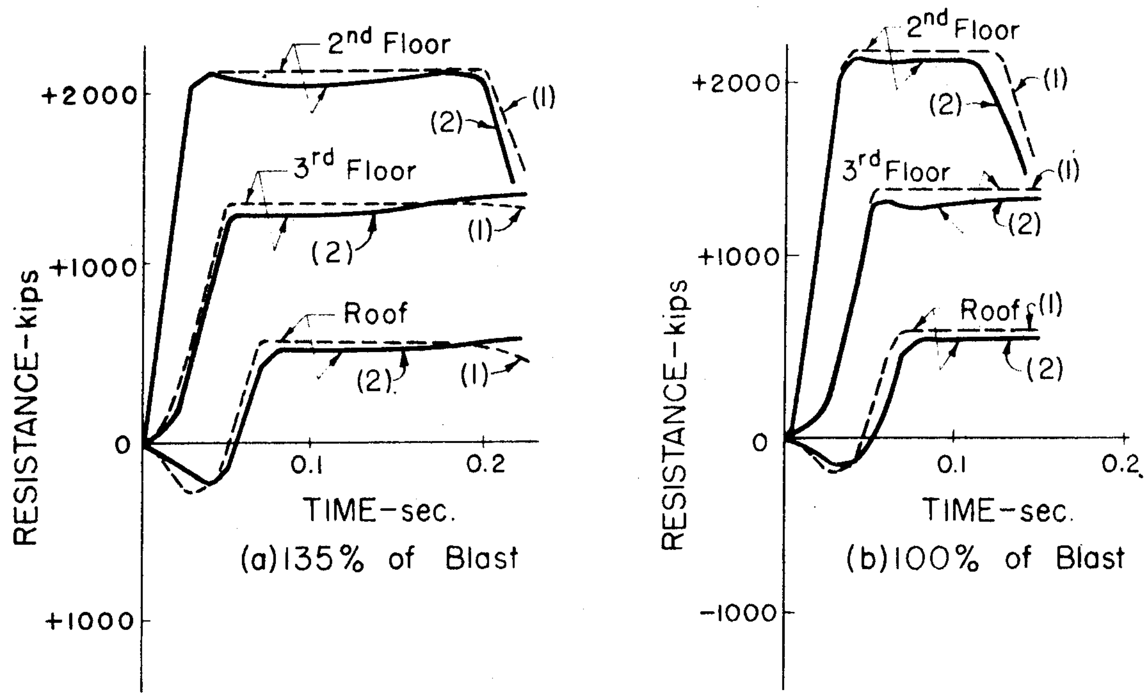


FIG. 2.4.5—80

[redacted]

For purposes of comparison, the constant plastic resistance used for preliminary frame designs and the final resistance of the steel frame building under 100% and 135% of the theoretical blast pressures are shown in figure 2.4.5-81



RESISTANCE CURVES—STEEL FRAME BUILDING No. 2

(I) Developed resistance from assumed resistance functions for preliminary analysis.

(2) Developed resistance from calculated resistance functions for final design.

FIG. 2.4.5-81

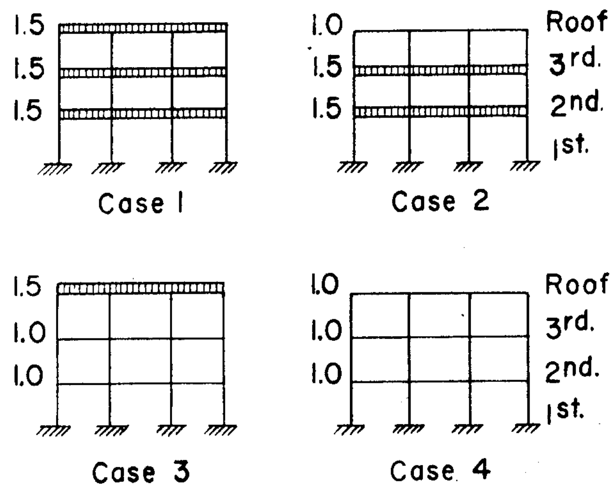
The use of an average constant resistance curve in the design of blast resistant structures seems to be justified by the results of these studies.

5. Effect of Variation of Mass

The sensitivity of the frames to change in mass is illustrated by the results of the four studies, Cases 1 to 4 in figure 2.4.5-82. From these results it is possible to generalize to some degree about the behavior of comparable three story frames; for example:

Cases 3 and 4 show that the increase in deflection of the top floor is a direct function of the mass added to the top floor.

A study of Cases 1 and 4 indicate that the increase in deflection of the first floor is inversely proportional to the mass added to all floors. In general a decrease in the relative mass of the roof will increase the relative deflection of the third floor and decrease the relative deflection of the roof.



Masses in case 2 are approximately equal to masses of concrete frame.

Superimposed load in all cases is approximately 100 lbs./sq. ft.

Figures beside floors are relative masses.

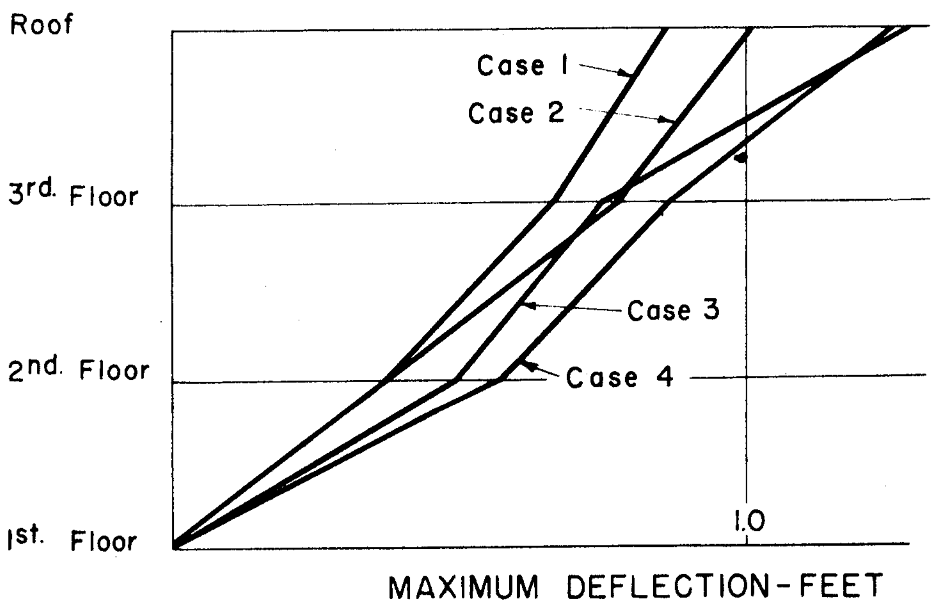
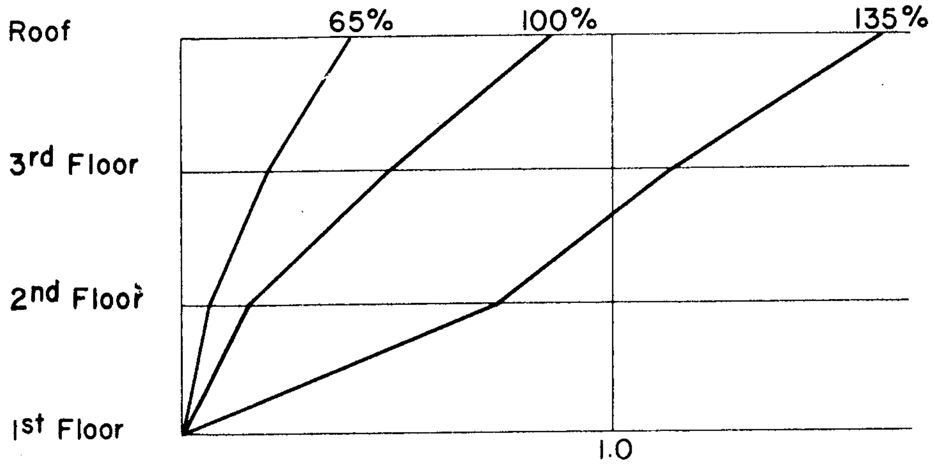


FIG. 2.4.5-82

[redacted]

6. Effect of Variation in Blast Load

The following studies illustrate the effect of blast load intensity on frame deflection. It is evident from the results shown in figures 2.4.5-83 to 85 that the percentage increase in deflection is much greater than the percentage increase in blast pressure load, and that the rate of increase in deflection is greatest in the lower floor.

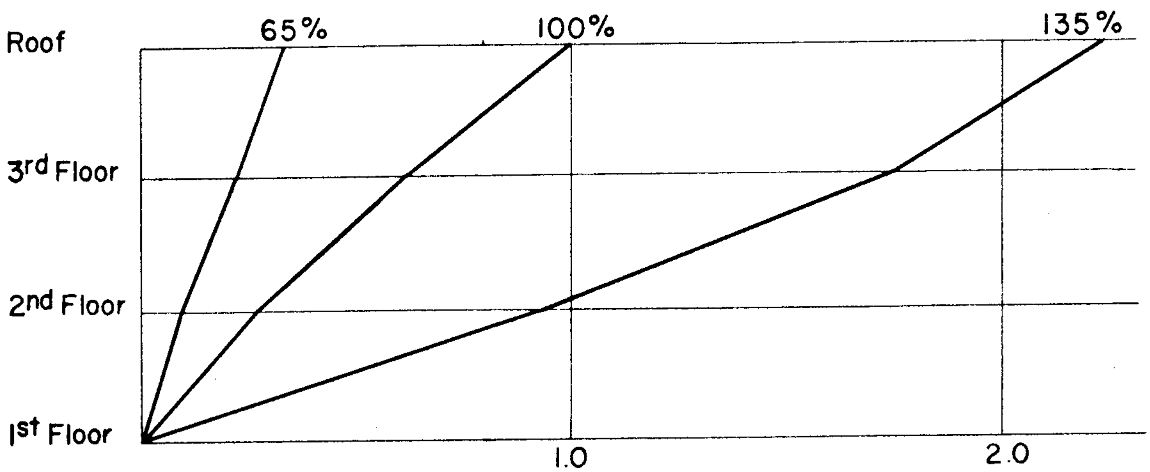


Series 1 MAXIMUM DEFLECTION – FEET

Steel Frame – Final Design – Non-participating Walls

Issue No. 5 Pressure Curves

FIG. 2.4.5-83



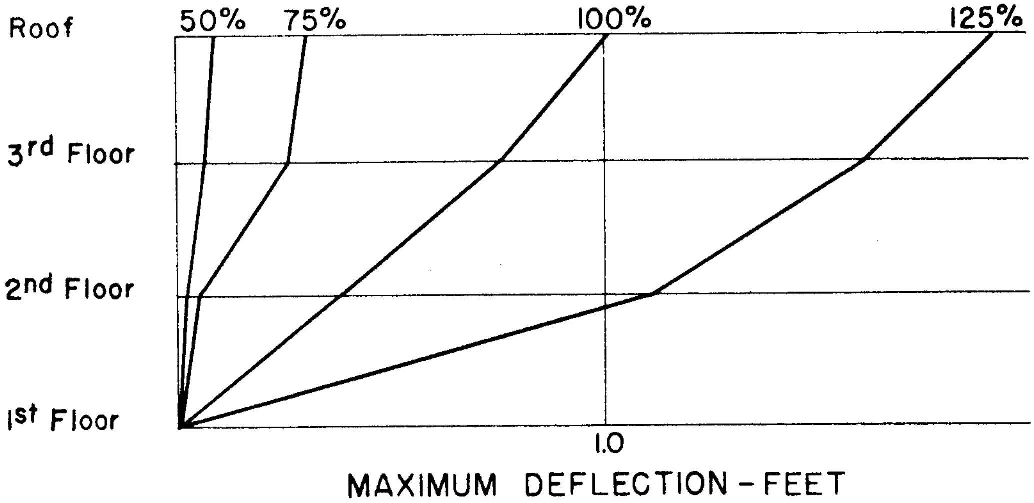
Series 2 MAXIMUM DEFLECTION – FEET

Concrete Frame – Final Design – Participating Walls

Issue No. 5 Pressure Curves

FIG. 2.4.5-84

[redacted]



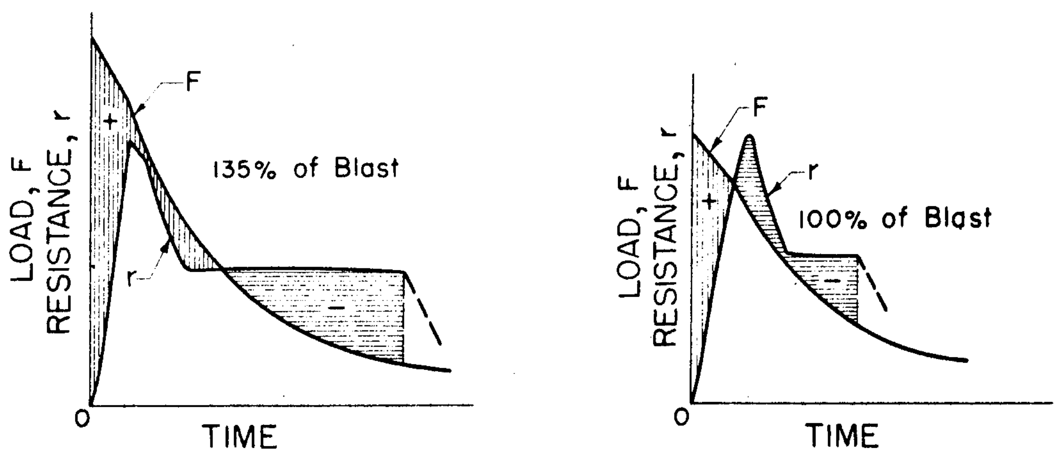
Series 3

Concrete Frame — Preliminary Design — Non-participating Walls

Issue No. 2 Pressure Curves — without rear wall pressure

FIG. 2.4.5—85

The effects of an increase in the blast intensity on the resistance and velocity are illustrated in figures 2.4.5-86 and 87 respectively.

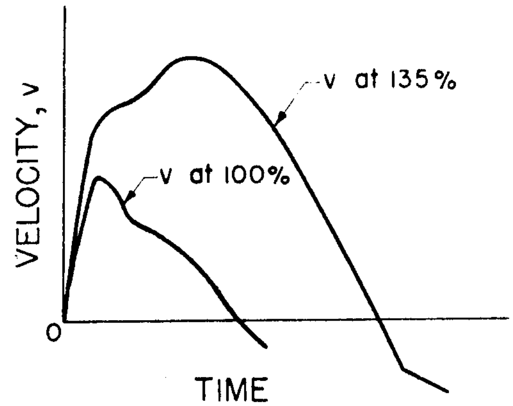


LOAD-RESISTANCE CURVES — VARIABLE BLAST

r = Net Resistance on 2^{nd} Floor Mass, Steel Frame

FIG. 2.4.5 - 86

[redacted]



VELOCITY CURVES–VARIABLE BLAST
2nd Floor Mass, Steel Frame

FIG. 2.4.5-87

An increase in the duration of the blast pressure also results in increased deflections. The illustration in figure 2.4.5-88 indicates the relation between resistance and load for this condition.

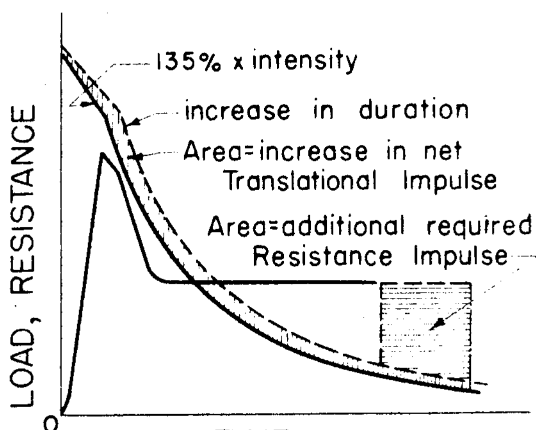


FIG. 2.4.5-88

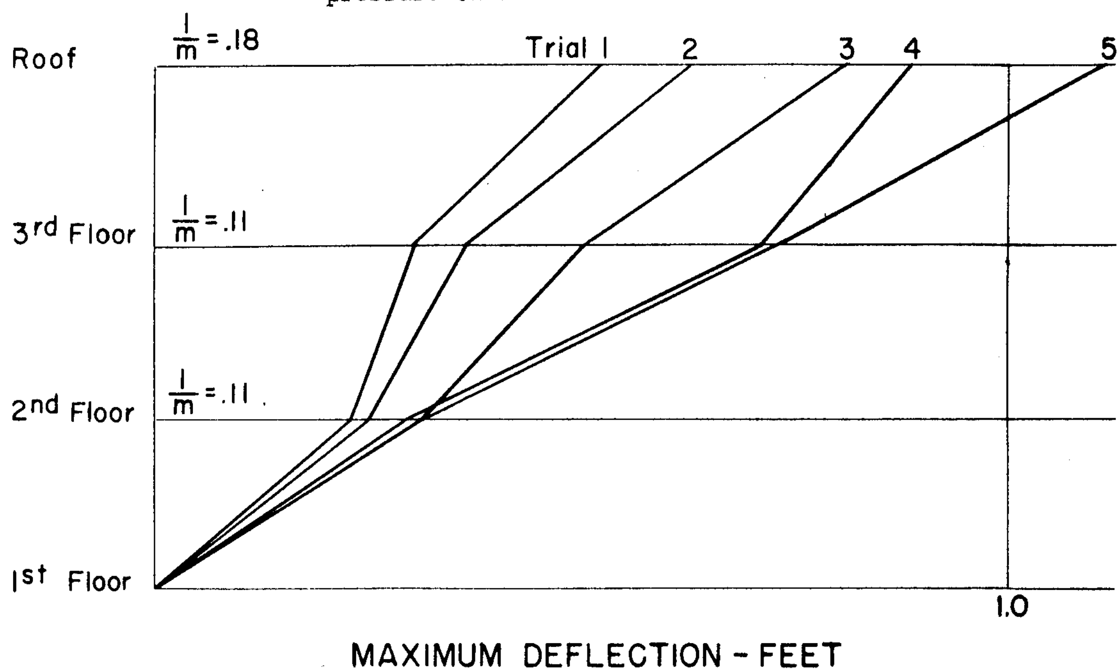
7. <u>Effect of Variation of the Relative Floor Resistances</u>

The column resistances were varied in a series of studies of frame action in which the additional variables were frame mass and blast pressure loading. The results shown in figures 2.4.5-89 to 94 were derived from these studies which include:

(Series 1) Concrete frame, non-participating walls, pressure on front and rear.

(Series 2) Concrete frame, non-participating walls, pressure on front only.

(Series 3) Steel frame, non-participating walls, pressure on front and rear.



Series 1

Non-participating Walls - Concrete Frame

100 % of Issue No. 2 Pressure Curves - with rear wall pressure

FIG. 2.4.5-89

For Series 1 the total shear r for plastic column moments between floors and the net resistances R at each floor level are shown in the table of figure 2.4.5-90. The quantities, $\frac{R_f}{R_2}$ and $\frac{R_2}{P}$ are also included, where

R_f = net plastic resistance at any floor level

R_2 = net plastic resistance at the second floor

P = peak blast load on second floor mass

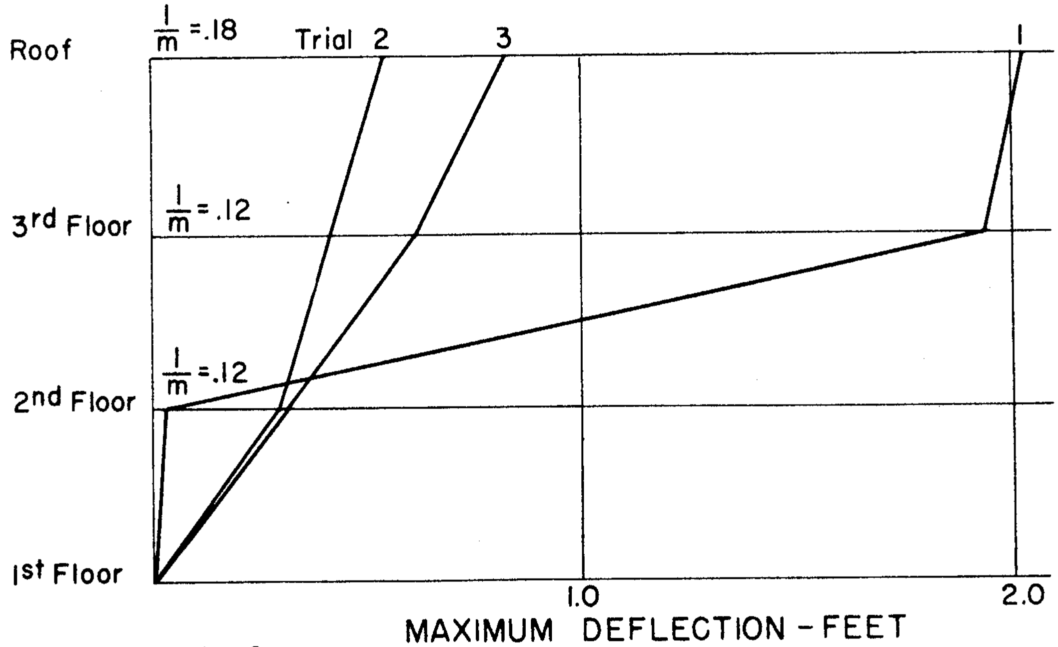
[redacted]

	Trial	1	2	3	4	5
r^{kips} =	Roof	135	100	75	75	65
	3^{rd} Floor	400	320	255	200	195
	2^{nd} Floor	750	640	550	500	485
R_f^{kips} =	Roof	135	100	75	75	65
	3^{rd} Floor	265	220	180	125	130
	2^{nd} Floor	350	320	295	300	290
$\frac{R_f}{R_2}$ =	Roof	.37	.31	.25	.25	.24
	3^{rd} Floor	.76	.69	.61	.45	.48
	2^{nd} Floor	1.00	1.00	1.00	1.00	1.00
$P = 1480^{kips}$; R_2/P =		.24	.22	.20	.20	.20

FIG. 2.4.5-90

The forces which resist the applied frame loads are the net column shears above and below the floor levels. Since the columns adjacent to a floor level may have about the same strength, a minor change in either, which may be a small fraction of the total resistance of a member, may produce a large percent change in the resistance of the members and therefore a material change in the net resisting force. This fact is evident from the information given in figure 2.4.5-89, where the increase in the relative deflection of the 3rd floor from trial 1 to trial 5 is due to this phenomenon. In this case a 50% reduction in net resistance causes a 700% increase in deflection. For the roof it is shown in Cases 4 and 5 that a 13% reduction in the resistance results in a 220% increase in the deflection. The behavior of the frame is complex and although these particular examples should not be taken as typical the sensitivity of the frame to changes in resistance is evident.

The results of Series 2, shown in figure 2.4.5-91 and 92, indicate similar sensitivity. It is also apparent that a change in the relative resistance of one floor greatly affects the behavior of the floors above and below.



Series 2

Non-participating Walls - Concrete Frame

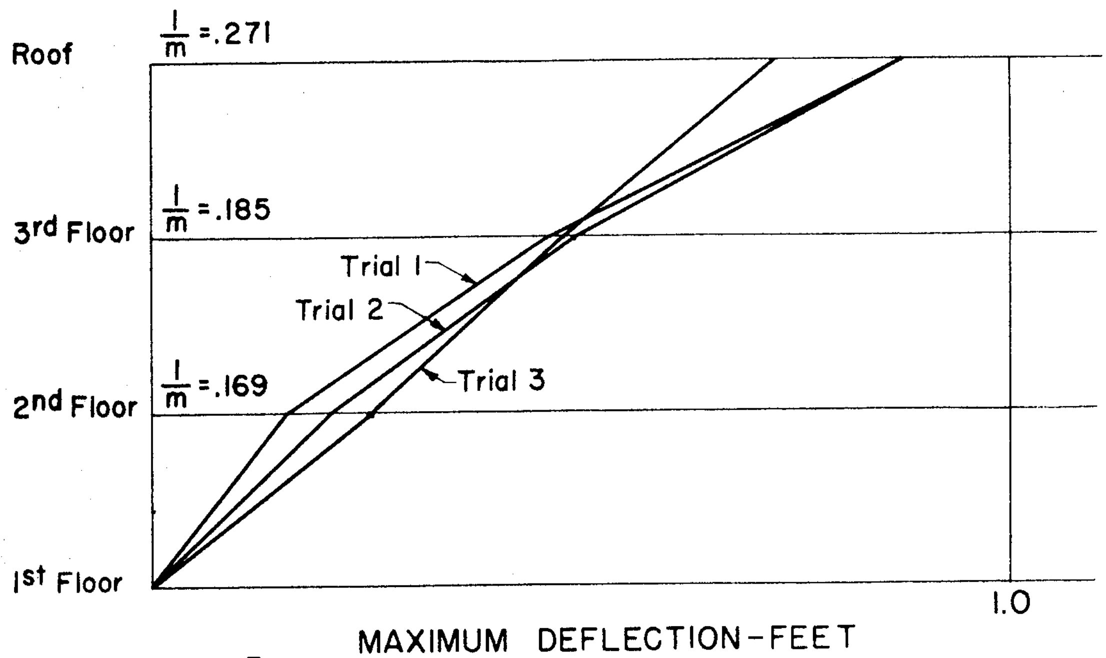
100% of Issue No. 2 Pressure Curves—without rear wall pressure

FIG. 2.4.5 - 91

	Trial	1	2	3
r^{kips}	Roof	100	190	150
	3rd Floor	250	490	410
	2nd Floor	840	840	765
R_f^{kips}	Roof	100	190	150
	3rd Floor	150	300	260
	2nd Floor	590	360	355
$\frac{R_f}{R_2}$	Roof	.17	.54	.42
	3rd Floor	.25	.86	.73
	2nd Floor	1.00	1.00	1.00
$P=1480^{kips}$; R_2/P		.40	.24	.24

FIG. 2.4.5-92

[redacted]



Series 3

Non-participating Walls — Steel Frame

100% of Issue No. 5 Pressure Curve — with rear wall pressure

FIG. 2.4.5-93

	Trial	1	2	3
r^{kips}	Roof	450	500	600
	3rd Floor	1300	1300	1400
	2nd Floor	2300	2200	2200
R_f^{kips}	Roof	450	500	600
	3rd Floor	750	800	800
	2nd Floor	1000	900	800
$\frac{R_f}{R_2}$	Roof	.45	.55	.75
	3rd Floor	.85	.89	1.00
	2nd Floor	1.00	1.00	1.00
$P = 1570^{kips}$; R_2/P		.64	.58	.51

FIG. 2.4.5-94

[redacted]

The behavior of the frame in series 3, shown in figures 2.4.5-93 and 94, is consistent with the results of lines 1 and 2. Differences between series 1 and 2 or 3 in mass and pressure necessitate the use of greater resistances in series 3. It is interesting to note that the deflections in any of the trials of this series might be acceptable since the range of results is quite limited. The first estimate of column resistance was based on experience gained in the analysis of similar frames.

If the loading conditions and masses are similar to those used for the test structure, the ratio of net resistances shown in figure 2.4.5-92 may be used as the first trial for other designs.

8. Simplified Frame Analysis

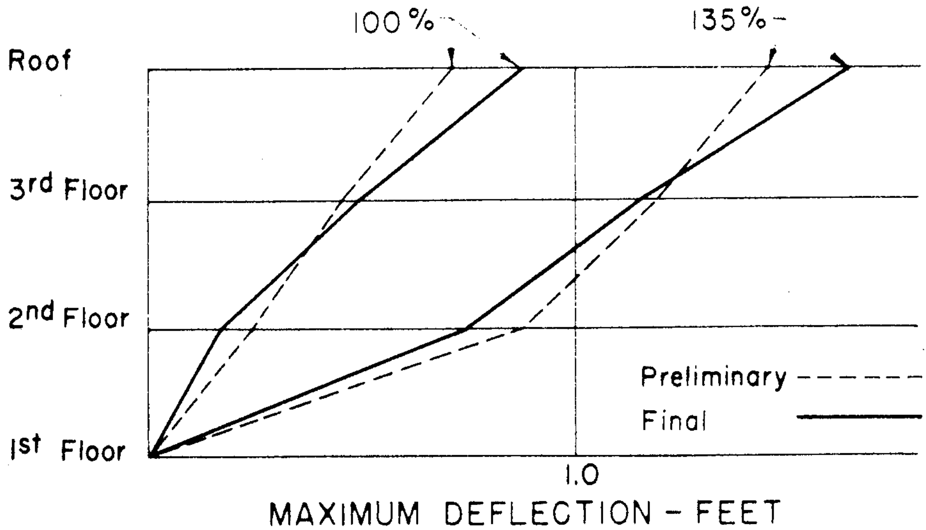
The following discussion is concerned with certain simplifications in frame analysis which were utilized in designing the non-participating wall type steel test structures. The reduction of labor in the analysis and design of structures with participating walls is also discussed and a method for achieving this purpose is suggested. The simplifications to be introduced in this case are dependent on further study of the effect of continuous walls.

a. Frames with Non-Participating Walls

Using the numerical procedure outlined for Case I (a) in Section 2.4.5-C3, the values of constant plastic resistance for each floor were varied; three trials being required to obtain the desired deflections. In these computations the story masses were based on the expected dead and live load.

The structure was proportioned on the basis of the results of the third trial. Clear column heights were assumed and column reactions were chosen so that the required story resistances would be developed for the range of axial loads prevailing during frame deflection. The remainder of the frame was designed on the basis of the worst conditions likely to be encountered with these column reactions.

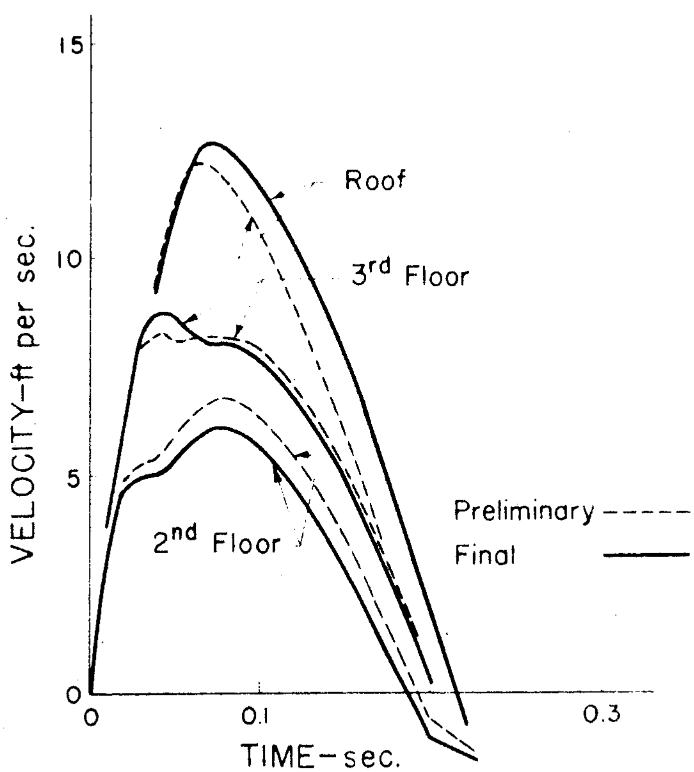
The frame was then analyzed using the procedure explained in Section 2.4.5-C3. The deflection and velocities obtained in the preliminary and final analyses are shown in figures 2.4.5-95 and 96.



Steel Frame

Preliminary and Final Analyses for Building 2.

FIG. 2.4.5-95



VELOCITY CURVES-STEEL FRAME AT 135% OF BLAST

FIG. 2.4.5-96

[redacted]

Circumstances peculiar to this particular case resulted in somewhat different clear heights for the third floor and roof columns in the final design than had beem assumed in the preliminary analysis. For this reason it may be expected that in the usual case the comparison of the preliminary and final analyses would be closer than indicated by these results because details of construction can be settled before the preliminary analysis.

The three trial analyses for the steel frames including the selection of the column sections used in the final design and analysis, were completed in less than 12 hours by one person. The close agreement of the preliminary and final analyses strongly suggests that the final analysis might be omitted entirely except as a check. Until more reliable data on the pressure loads and on the behavior of materials under rapid loading are obtained, an exhaustive analysis may not be warranted in view of discrepancies in the accuracy of the basic data.

b. Frames with Participating Walls

(1) Continuous Walls Restrained at Each Floor Level

In this case the procedure for frame design is similar to that described for the case of frames with non-participating walls except that the resistance function must be altered to account for the differences in column and wall flexibility in the elastic range.

(2) Continuous Walls Supported At Each Floor Level

At this writing it seems probable that by comparing the required column resistances for frames with and without participating walls, but otherwise similar, a fairly reliable rule of thumb procedure for obtaining the first trial values of column resistance can be evolved. The behavior patterns of frames with and without participating walls are not exactly the same but in each case the patterns are so definite that it seems likely that the effect of the wall can be expressed as an equivalent column resistance. Therefore, the following method, illustrated for a typical concrete frame, for determining empirical rules to evaluate the effect of the walls may prove feasible.

(a) By a simplified preliminary analysis, assuming non-participating walls, determine the net resistance required at each floor level to maintain the deflection within the specified limits.

(b) Having designed the walls to resist the local blast pressure, determine their equivalent resistance to lateral motion.

To establish a criterion for the relative effectiveness of the walls compared to the columns in resisting deflection, it is necessary to account for the time distribution of the resistance impulse.

[redacted]

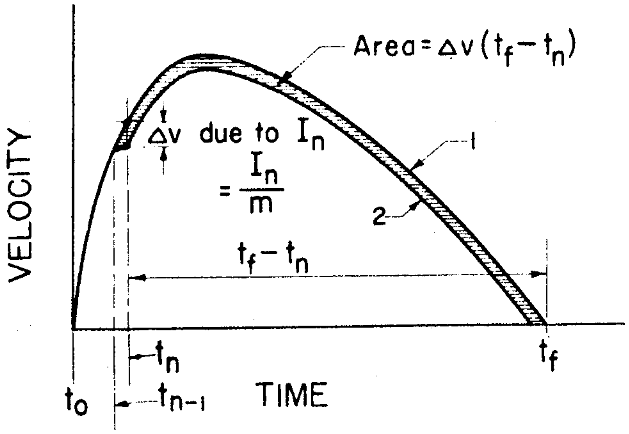


FIG. 2.4.5-97

In figure 2.4.5-97 velocity curve 2 differs from curve 1 only in that from time, t_{n-1} , to time, t_n , an additional resisting impulse, I_n , produced the difference in velocity, Δv .

The difference in deflection by curves 1 and 2, then is $\Delta v(t_f - t_n)$. It is evident that the greater the time $t_f - t_n$, the more effective will be the impulse I_n in reducing the deflection. Relations between the column and wall resistances for a typical frame with participating walls designed to resist the local pressure loads and with continuous positive steel are shown in the table of figure 2.4.5-98. The following notation is used;

I_c = resisting impulse supplied by columns

I_w = resisting impulse supplied by walls

t_i = time at which resisting impulse is applied

t_f = time at which motion of the frame is complete

r_c = approximate average column shear of the columns below the floor

r_w = approximate ultimate wall shear of the wall below the floor which could be developed if the walls were fixed at each floor level

The signs, + and - indicate impulse in the direction of and opposite to the direction of the blast load respectively.

The quantities, $\left[I_c(t_f - t_i)\right]$ and $\left[I_w(t_f - t_i)\right]$, are a measure of the relative effectiveness of the actual resisting impulse produced by the columns and walls respectively. The ratio, r_w/r_c , indicates the ultimate strength of the walls relative to the columns if the walls were fixed at the floor levels. All summations are made over the interval, $t_f - t_o$.

1	2	3	4	5	6	7	8
Floor	ΣI_c	ΣI_w	$\frac{\Sigma I_w}{\Sigma I_c}$	$\Sigma[I_c(t_f - t_i)]$	$\Sigma[I_w(t_f - t_i)]$	$\frac{\Sigma[I_w(t_f - t_i)]}{\Sigma[I_c(t_f - t_i)]}$	$\frac{r_w}{r_c}$
2	-3454	-704	0.20	-1190	-760	0.64	0.8
3	-1415	-1398	0.99	-485	-431	0.89	1.5
Roof	-1506	-74	0.05	-338	+283	-0.84	1.0

FIG. 2.4.5-98

Although it appears that the wall tends to increase the deflection of the roof, any positive impulse supplied by the wall to the roof is accompanied by an equal impulse of opposite sense applied to the 3rd floor. In general wall action tends to equalize the relative frame deflections. The figures of columns 4 and 6 reflect this behavior.

Not enough work has been done to formulate any but the roughest rule of thumb for estimating the required column sizes in a building with participating walls. The maximum wall resistance might be computed by assuming that the walls are fixed at each floor level and act as columns. From the information given in the preceding table, criteria may be developed by which the effective wall resistance in each story may be computed, i.e., the wall actually furnishes an effective resistance equal to the following percentages of the maximum resistance.

Second floor walls, 80%

Third floor walls, 60%

Roof Walls, 80%

These values were determined by dividing the relative impulse-duration function of the walls (column 7) by the relative strength of the walls (column 8).

(c) Determine the required resistances to be furnished by the columns. This will be equal to the total required resistance less the resistance provided by the walls.

[redacted]

9. Conclusions

The numerical methods developed in Section 2.4.5-C for the analysis of frames are generally satisfactory, the accuracy of the basic analysis being controlled primarily by the size of the time intervals. The behavior of the frames is such that large variations may result from small discrepancies in applied loads and frame resistances.

Floor	W[1] (kips)	Σ W	Ultimate Shear[2] per Story (columns only) (kips)			Ratio $\frac{\text{Story Shear}}{\text{Roof Column Shear}}$		
			Wind[2,3]	Earthquake[2,4]	Blast	Wind	Earthquake	Blast
Building 2 (Steel Frame)								
Roof	119							
		119	19.4	22.4	550	1.00	1.00	1.00
3rd	178							
		297	57.5	55.8	1275	2.96	2.49	2.32
2nd	192							
		489	96.5	91.8	2000	4.98	4.10	3.64
1st	—							
Building 3 (Concrete Frame)								
Roof	202							
		202	19.4	38.0	250	1.00	1.00	1.00
3rd	288							
		490	57.5	92.0	650	2.96	2.42	2.60
2nd	304							
		794	96.5	148.0	950	4.98	3.90	3.80
1st	—							

FIG. 2.4.5-99

1. W = weight per story = DL + LL (LL = 100 lbs. per sq. ft.).
2. Ultimate shear based on normal design practice for wind and earthquake loads.
3. Wind load = 90 lbs. per sq. ft.
4. Earthquake load = $0.1\,\Sigma W$.

[redacted]

There are strong indications that many simplifications in the frame analysis can be made without an appreciable reduction in the accuracy of the solution, but the formulation of these procedures must await the post-test analysis for further information concerning the effects of the numerous variables.

Blast resistant structures may be compared to earthquake resistant structures which are usually designed by the method of equivalent lateral loads, the magnitudes of which are proportional to the total mass. In contrast, when designed for blast loads, heavier buildings require less blast resisting strength than relatively light structures. In the table of figure 2.4.5-99 the theoretical column shears required by steel and concrete frame buildings for earthquake and wind loads and A-bomb blast loads are tabulated.

An attempt to use earthquake resistant structures as a guide for determining relative floor strengths could not take into account the variations in intensity of the reflected blast pressures, the shape and orientation of the building, and the effect of window-openings, each of which was found to produce a major effect on the necessary blast resistance. The value of the earthquake designs appears to be limited to that of a guide for the design of joints and connections.

E. Overturning Stability

The initial force applied to structures by A-bomb blasts is a high reflected pressure on the exposed face of the structure tending to overturn the building. A short time elapses before the pressure begins to act on the rear wall. In the interim the roof begins to receive blast loads. The forces which act on the roof and rear wall assist the force due to the weight of the structure in preventing overturning.

The stabilizing pressures on the roof and rear wall are important in that a change in either results in a different rotation about the rear footing or in certain cases may even result in instability. Therefore information concerning the area affected by and the nature of the edge disturbance is needed because this phenomenon is an important factor in estimating overturning stability.

Structural behavior in resisting overturning can be generally classified into two types, (1) relatively flexible frame action which can transmit overturning forces only in proportion to the shear and moment capacity of the frame members, and (2) the shear-wall type of structure which rotates essentially as a rigid body under the action of the blast loads undergoing angular acceleration about the rear base. The latter case is usually more critical and will be explained in detail.

The roof load on the interior columns of typical shear-wall type structures is usually large enough to prevent the interior of the building from rotating relative to the rear base. Although the footings, floor system and roof framing act in flexure to resist the motion of the rigid surrounding walls, their strength and stiffness are generally insufficient to prevent front wall rotation relative to the first row of interior columns.

[redacted]

The angular motion of the exterior walls is caused by the unbalanced loads acting on the structure. An equation relating these loads to the resistance, mass and angular acceleration is presented below.

Let dm = mass of any particle in the rotating body

ρ = perpendicular distance from the axis of rotation of any particle

$\ddot{\theta}$ = angular acceleration of each particle about the axis of rotation

I_0 = polar moment of inertia of the rotating bodies about the axis of rotation

M = total moment about the axis of rotation of all the forces acting on the body

At any time $M = \int \rho^2 \ddot{\theta}\, dm$

since $I_0 = \int \rho^2 dm$

$M = I_0 \ddot{\theta}$

Because the pressures on the front face, roof and rear face vary with time and the shears and moments developed in the framing connecting the walls to the interior columns vary with deflection, the magnitude of the overturning moment is a function of both of these variables. To account for these variations a step-by-step numerical procedure was adopted using an average net moment for each step. This procedure is presented in the table of figure 2.4.5-100.

1	2	3	4	5	6	7	8	9	10
t	Δt	M	$\overline{M}$	$\ddot{\theta}$	$\Delta\dot{\theta}$	$\dot{\theta}$	$\dot{\theta}_{avg.}$	$\Delta\theta$	θ
t_0		—				—			—
	—		—	—	—		—	—	
t_1		—				—			—
	—		—	—	—		—	—	
t_2		—				—			—
.									
.									
t_{f-1}		—				—			—
	—		—	—	—		—	—	
t_f [1]		—				—			—
time	$\Delta t = t_n - t_{n-1}$	Computed moment at time t_n	$\overline{M} = \frac{M_n + M_{n-1}}{2}$	$\ddot{\theta} = \frac{\overline{M}}{I}$	$\Delta\dot{\theta} = \ddot{\theta}\Delta t$	$\dot{\theta}_n = \dot{\theta}_{n-1} + \Delta\dot{\theta}$	$\dot{\theta}_{avg.} = \frac{\dot{\theta}_n + \dot{\theta}_{n-1}}{2}$	$\Delta\theta = \dot{\theta}_{avg.}\Delta t$	$\theta_n = \theta_{n-1} + \Delta\theta$

1. Analysis completed at t_f

FIG. 2.4.5-100

[redacted]

The following comments, which explain the analysis during any time interval, Δt, are made to clarify the procedure.

M is the instantaneous summation of the effective moments, about the axis of rotation, due to the front, roof and rear wall pressures, and the weight of the structure. In flexible structure M is the overturning moment caused by axial loads, shears or moments in the framing.

$\overline{M}$ is the average of the total moments at the beginning and end of the interval.

$\ddot{\theta}$ is the average angular acceleration of the rotating mass during the time interval.

$\Delta\dot{\theta}$ is the change in angular velocity in the interval.

$\dot{\theta}$ is the total angular velocity at the end of the time interval.

$\dot{\theta}_{avg.}$ is the average angular velocity during the time interval.

$\Delta\theta$ is the rotation during the time interval.

θ is the total rotation at the end of any time interval.

The maximum angle of rotation $\theta_{max.}$, occurs when the angular velocity, $\dot{\theta}$, becomes zero. To obtain the maximum upward deflection of the front wall, the maximum angle of rotation, in radians, should be multiplied by the depth of the structure, measured parallel to the direction of the blast load. A limiting value for the front wall uplift will depend on the type of construction and on the damage which can be sustained without preventing continued use of the structure.

F. Sliding Stability

When the blast pressure wave reaches the front face of the structure the high reflected pressure tends to translate the building in the direction of the blast. After a very short time the pressure begins to act on the rear wall and helps to resist this motion. A typical translational force-time curve is shown in figure 2.2.1-5. Sliding of the building under this net blast load including the inertia forces caused by the horizontal component of the rotation, is resisted by friction and by the active and passive soil pressure acting against the back of the footings.

The frictional resistance depends on the properties of and on the pressure between the two sliding surfaces. For rough concrete on soil the coefficient of friction may be expected to be equal to $\tan\phi$, where ϕ is the angle of internal friction of the soil.

The total force exerted by the footing on the soil will be equal to the effective weight of the structure plus the blast pressure on the roof. For any particular time interval the roof load may be obtained from the roof pressure-time curves.

Sliding stability may be investigated by use of the step-by-step procedure using equal time intervals as illustrated in the table of figure 2.4.5-101.

t	Δt	W_e	P_r	$C(W_e+P_r)$	P_p	P_{net}	ΣF
t_0							0
	—	—	—	—	—	—	
t_1							—
	—	—	—	—	—	—	
t_2							—
⋮							⋮
t_f [1]							0
						$\Sigma(\Sigma F)$	—

1. Analysis complete at t_f

FIG. 2.4.5-101

The following notations, used in the step-by-step analysis, represent average values as indicated during any time interval:

W_e = effective weight of structure and contents undergoing translation

P_r = total vertical blast load on structure

P = total vertical load on the foundations including the inertia forces resulting from rotation = $W_e + P_r$

C = coefficient of sliding friction

P_{net} = net translational force on the structure including the effects of rotation

P_p = resistance due to passive pressure of the soil

F = unbalanced translational force where, $F = P_{net} - C(W_e + P_r) - P_p$

As shown below, the quantity ΣF at any time interval is proportional to the total velocity at the end of the time interval.

$$F = ma = m\frac{\Delta v}{\Delta t}$$

$$\Delta v = F\frac{\Delta t}{m}$$

$$v = \Sigma \Delta v$$

$$= \Sigma F\left(\frac{\Delta t}{m}\right)$$

[redacted]

where;

a = average acceleration of building during the time interval

m = total mass being accelerated

Δt = time interval

ΔV = change in velocity of the building during any time interval

V = velocity of the structure at the end of the time interval

Thus, when $\Sigma F\left(\frac{\Delta t}{m}\right) = 0$ the velocity is zero and the building has ceased its translatory motion. If $\frac{\Delta t}{m}$ is kept as a constant throughout the computation the velocity becomes zero when $\Sigma F = 0$, and the quantity $\Sigma[\Sigma F]$ is proportional to the total displacement.

The increment of displacement in each time interval will then be

$\Delta \delta_h = \frac{\Sigma F \Delta t^2}{m}$ and the total displacement, $\delta_h = \Sigma \Delta \delta_h$

$$\text{or } \Sigma \Delta \delta_h = \frac{\Delta t^2}{m} \Sigma[\Sigma F]$$

where; $\Delta \delta_h$ = change in displacement of the structure during any time interval

δ_h = displacement at the end of any time interval

For structures with deep foundations, basements, etc., the possibility of sliding is small because of the large passive earth pressure which can be developed to balance the net blast load. In the case of buildings with shallow foundations similar to the test structures the various factors necessary for an investigation of sliding stability can be computed with reasonable accuracy.

G. Roof Trusses Under Blast Pressure Loads

1. Introduction

The magnitude of stresses produced in a truss is a function of the shock waves propagated in the various members. It is evident that two or more shock waves emanating from separate sources will reinforce each other at certain points. Because each truss member vibrates in a different mode according to the natural frequency and load, the calculation of these stresses is very difficult, if not impossible. However, their occurrence should be recognized and an attempt should be made in the design to minimize their effects.

[redacted]

Theoretically, shock wave fronts will have less chance to build up to a yield point in a single member if pin joints are used. Conversely, stiff joints produced by welding should be avoided. Riveted connections, subject to a slight amount of yielding, are better than welded joints. The damping effect of yielding, particularly in tension members, should be utilized when possible.

2. Loads on Truss

Vertical forces are transmitted to the truss purlins which in turn support the response of the roofing. A typical relationship between these forces for light weight roofing is shown in figure 2.4.5-102a. It is assumed that the covering is elastic and vibrations are neglected in accordance with the concept that the net impulse due to an oscillating load may be accurately represented over a sufficiently long time period by the average impulse.

In figure 2.4.5-102b a similar relationship is shown for heavy roofing. In this case the roofing can be relied on for plastic resistance after which the roof slab vibration is also neglected. For the case of heavy or light roofing the purlin vibration after plastic resistance ceases is also neglected to obtain the truss loading.

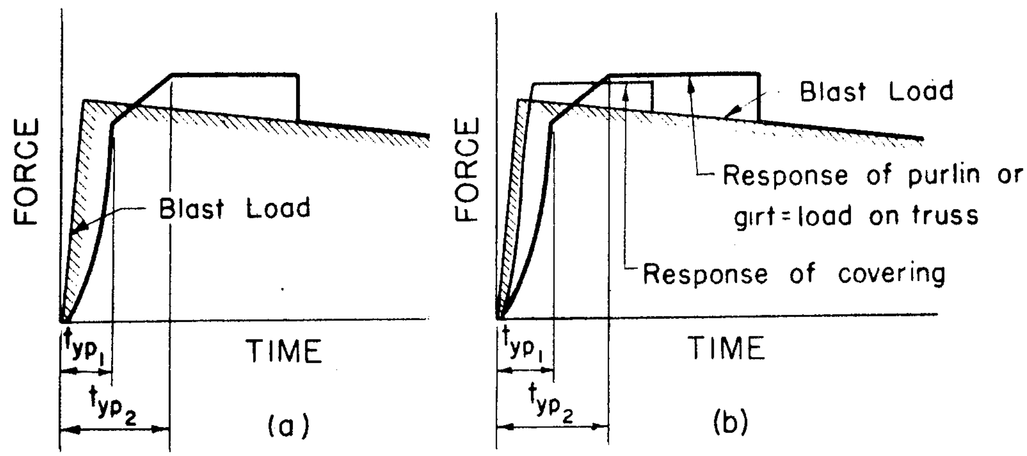


FIG. 2.4.5-102

Since most of the equivalent mass of the frame is concentrated at the truss level, the chords and diagonals act to transfer the force producing acceleration to the mass. The net horizontal thrust at any time can be computed by superposition of the forces from the front and rear wall on the total column resistance. The maximum net value of horizontal force occurs at the instant the blast pressure begins to act.

In addition to vertical load and horizontal thrust, the column moments must be resisted. These moments are a function of deflection and thus take time to build up to maximum values. If the front col-

[redacted]

umns are subjected to bending by local wall loads the moment produced in these members at the truss level will in most cases be reversed by deflection of the frame.

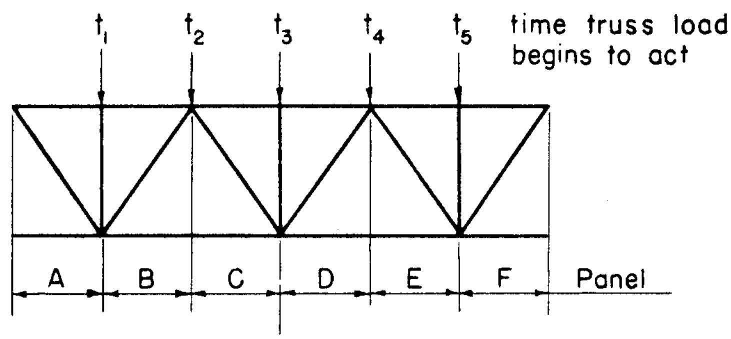
3. Design Procedure

It is evident that in order to be practical, a design method must omit quantitative internal shock wave considerations. The sizes of diagonals and struts will be governed by the vertical and horizontal load while the chords depend on a combination of vertical and horizontal load and column moments. In designing a member for these loads the rate of loading must be used to obtain probable yield point stresses from the recommended design curve shown in figure 2.5.2-1.

The variation of the forces described above may be determined by treating the loads separately with respect to time and superimposing the results.

a. Vertical Loads

For illustrative purposes the truss shown in figure 2.4.5-103 is discussed. Assuming for a first trial that the natural period of the proposed truss is such that the truss response will follow the loading very closely, the vertical shear as a function of time can be calculated. The method of recording computations shown in figure 2.4.5-104 is suggested.



Truss loaded by reactions of purlins

FIG. 2.4.5-103

Time	Load P_1 between panels					Shear in panel					
	A,B	B,C	C,D	D,E	E,F	A	B	C	D	E	F
t_1	—	—	—	—	—	—	—	—	—	—	—
t_2	—	—	—	—	—	—	—	—	—	—	—
.											
.											
t_f [1.]	—	—	—	—	—	—	—	—	—	—	—

1. Analysis completed at t_f

FIG. 2.4.5-104

[redacted]

b. Horizontal Loads

As stated previously the maximum net value of horizontal thrust causing acceleration in the truss occurs in the first instant when the front pressure is a maximum and column resistance is zero. The amount of mass that is concentrated at the truss level for purposes of analysis is in most cases somewhat greater than the mass which is actually dependent on the truss for its acceleration. As can be seen in figure 2.4.5-105, the front wall mass is accelerated by forces applied directly to the wall, not through the truss. Therefore, trusses designed for the total force producing acceleration will be proportioned on the side of safety.

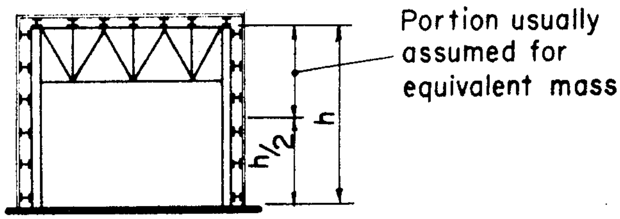


FIG. 2.4.5-105

If the truss mass and roof mass are rigidly connected, the net force remaining in the diagonals and chords at any point will be an inverse function of the mass up to that point. With the column resistances known as a function of time either by estimates or by frame analysis, it is possible to determine the horizontal thrust as a function of time. On the basis of these forces, it is possible to compute the approximate stress in each truss member as a function of time.

4. Design of Truss Members

a. Tension Members

If longitudinal oscillation effects are neglected the design of tensile diagonals involves only the determination of the proper yield stress based on the rate of loading. Yield point stresses in tension members may be caused by the load or by two or more shock waves reinforcing each other. This phenomenon of yielding would seem to be a welcome factor for the truss as a whole.

b. Compression Members

The result of a theoretical investigation (32) into the action of pin-ended columns under an impact load of constant magnitude is shown in figure 2.4.5-106. It can be seen that for a constant pulse less than Euler's load the response is never greater than two. The pins were assumed to be held by rigid, non-yielding supports.

(32) C. Koning and J. Taub, "Impact Buckling of Thin Bars in the Elastic Range Hinged at Both Ends."

[redacted]

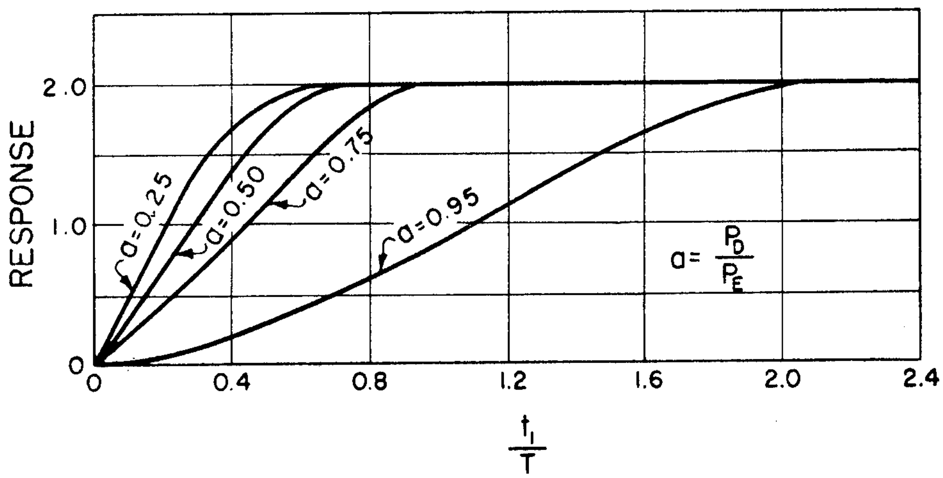


FIG. 2.4.5-106

The terms used in figure 2.4.5-106 are defined as:

P_D = magnitude of impact load

P_E = Euler's load

t_1 = duration of impact under constant pulse

T = natural frequency of member for transverse vibration

Response is the ratio of stress produced by P_D to stress produced by a static load of equal magnitude.

Truss members which are acted on by loads requiring finite time in which to build up from zero to a maximum, undergo a much less severe type of loading than that used in obtaining the results shown. Therefore if compression members are designed in the elastic range using the peak blast load as a constant pulse, the actual maximum stresses will not exceed the design value.

Since further information pertaining to the problem is lacking, the following design procedure for truss members is proposed. Conservative results will be obtained but these are at least partially justified in view of the neglected shock waves.

(1) with compression members stressed within the elastic range;

(a) Based on the requirement, $\frac{P_E}{P_D} > 1$, determine a trial member.

(b) Determine the maximum stress using P_D as a static load. It may be advisable in some cases to assume an initial crooked-

ness based on rolling mill practice.

(c) Compute the unit stress based on a response taken from figure 2.4.5-106. The maximum response should not exceed the yield point of the material.

(2) With tension members stressed beyond the elastic range;

(a) and (b) The first two steps of the design would be the same as for steps a and b for case 1 above.

(c) If one or both ends of the compression member is supported by other members undergoing plastic strains, the response factor of the compression member will be reduced, the magnitude being governed by the response of the connecting members.

H. Design of Reinforced Brick Work

The American Ceramics Society through its Committee on Reinforced Masonry conducted a testing program in the 1930's to determine the flexural strength of brick slabs and beams reinforced with steel rods placed in the mortar joints.

The test slabs were composed of a single layer of brick set on edge with the long dimension of the brick in the direction of the slab span. The panels spanned 8 feet between supports and cantilevered 4 feet beyond the supports at each end. The positive reinforcing steel in the center span consisted of one 3/8" Ø bar per joint placed one inch above the bottom. The negative steel was also one 3/8" Ø bar per joint placed one inch below the top of each side span. The negative steel extended to the 1/4 point of the center span. The steel percentage was slightly under 1% of the total slab area. All the tests were conducted using static loads; all the slabs were one-way; and none of the panels were tested for either reversed or repeated loading.

Disregarding the small number of slabs with obvious joint weaknesses caused by low quality mortar, incompletely filled joints, and poor adhesion between the brick and mortar, the failures were of a ductile nature and were caused by yielding of the steel. Computed bond stresses were in general equal to or greater than 200 p.s.i. No failures occurred due to compression in the brick. Judging from the measured deflection of the test panels, it appears that the same limiting deflection criterion of L/32 used for the concrete members is conservative for use in the design of reinforced brick panels.

Brick walls may then be designed using the same design methods and the same design criteria used for the one-way reinforced concrete panels. However, it may be expected that the brick panels will be much more erratic in behavior than the concrete panels and some panels may fail at loads below the expected design strength.

2.4.6 Soil Resistance under Dynamic Loads

In the design of the footings, the condition of momentary high loads on the rear footings of the buildings must be considered. Depending on the height and weight of the building and the blast pressures on the front, roof and rear faces, as discussed in Sections 2.4.5-E and F, these increases in footing pressures may be as much as five to ten times the conventional dead and live loads and the footings must be designed on this basis.

The conventional design criteria may be modified for this temporary load condition as loads of the intensity being considered will necessarily cause considerable cracking and deformation of the buildings with the result that larger differential movement is tolerable under blast loading than would be the usual case. However, the footing settlements should not be so large as to reduce the effective blast resisting capacity of the structure.

The duration of these high footing pressures is extremely short and dynamic load tests (33) show that certain types of soil increase in strength and stiffness under rapid loading. In these tests the loads were applied in times varying from 0.02 seconds to 10 minutes with the following results:

(1) The shearing strength of clays and shales was increased by approximately 50% to 100%

(2) The shearing strength of sand was increased approximately 10%

(3) The modulus of deformation of the clay and shale was also increased approximately 100%

(4) The modulus of the sand was practically independent of the rate of loading.

2.4.7 Effect of Live Load

The important effect of the mass on the frame strength necessary to limit the lateral deflections within the desired limits led to the inclusion of a portion of the design live load as part of the test load of Buildings No. 2 to No. 6.

Many practical difficulties arose in obtaining a type of loading which would be both representative of the actual live load and easy to remove.

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(33) A. Casagrande and W. L. Shannon, "Strength of Soils under Dynamic Loads"

[redacted]

In the design analysis this live load is considered as acting integrally with the floor mass. To assure such integral action it was earlier suggested that the live load should be formed of precast concrete blocks bolted directly to the floor; however this proposal was rejected on the basis of added cost and difficulty expected in removing such units for the later inspection of the floor.

In view of this, the 100 p.s.f. live load was finally simulated by bags filled with coral sand. These bags are to be evenly distributed over the various floor areas except in certain small areas which are left exposed to clear the supports of the instruments. To prevent the sand bags from moving excessively under the impact of the blast pressure on the building, they are stacked within a network of timber members which are on 4 foot centers in directions parallel to and perpendicular to the front face of the buildings. The timber members are in turn anchored to the floor slabs by short lengths of structural steel shapes embedded in the concrete.

The position and condition of the sand bags should be examined and recorded before and after the test to assure the correctness of assuming integral mass action. In addition, instrumentation will be provided to keep a continuous record of any relative motion of the live load during the motion of the test structures. If post-test examination shows that the live load has moved with respect to the floor, then the effect of the delayed action of the simulated live-load mass both under acceleration and deceleration must be evaluated.

2.4.8 Shear in Reinforced Concrete Members

The need for additional information on the ultimate shearing strengths of concrete members is evident from even a cursory study of the present status of this phase of the design. While the information available on the shear behavior in members under static loads is meager, little or none is available for members under dynamic loads and further tests would certainly be useful in narrowing the necessarily broad limits of uncertainty assumed in the design of the test buildings.

In the designs for the test structure an ultimate shearing strength of from 6% to 10% of the compressive strength of the concrete, as measured by diagonal tension, (34, 35) was expected for members without shear reinforcement. For 3000 p.s.i. concrete this meant a variation between 180 to 300 p.s.i. and sample members were included which cover this range, as well as a few members at higher and lower values. As these stresses are slightly higher than the 150 p.s.i. shear stresses successfully carried in the beams tested under a dynamic load by the Massachusetts Institute of Technology,(7) important members, such as walls and panels which should

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(34) F. E. Turneaure and E. R. Maurer, Principles of Reinforced Concrete Construction

(35) W. A. Slater, A. R. Lord and R. R. Zipprodt, "Shear Tests of Reinforced Concrete Beams"

(7) J. B. Wilbur, R. J. Hansen and K. Steyn, "Behavior of Reinforced Concrete Structural Elements Under Long-Duration Impulsive Loads"

[redacted]

not fail before the frame receives the full impulse of the loading, were limited to a unit shear of 250 p.s.i. unless adequate web reinforcement was provided.(36) The members of the different parts of the test structure were varied to determine the effect of the jd, the percentage of tensile steel, and the magnitude and distribution of the compression stresses in the concrete.

The high intensity of the reflected pressures led to the prolific use of web reinforcement in the front and rear walls of test Buildings No. 3 and No. 4. As such reinforcement is expensive and difficult to handle the behavior of the test panels without this web reinforcement should be carefully studied to find the upper limits below which these bars may be eliminated.

Available data would indicate that an ultimate diagonal tensile strength, as measured by the shearing-stress, of from 1000 to 1800 p.s.i. may be developed by use of vertical web steel. Turneaure states that 20% of the concrete strength may be readily developed and 30% may be developed by careful design. Without substantiating data it might be presumed that some further increase may be expected because of the dynamic nature of the loading.

In the design of the test buildings the shearing stresses of members with shear reinforcement were limited to ultimate unit shearing stresses of (34, 35)

$$v = \frac{1}{1.33}(0.005\, f_v + r\, f_v)$$

and were computed on the basis of the total shear which existed at a distance from the support equal to one-half the depth of the member,

where r is the percentage of web reinforcement

f_v = 50,000 p.s.i. = stress in web steel

and $v = \dfrac{V}{b\,j\,d}$

and 1.33 is a factor of safety.

The members were usually of such a size that the unit stresses are below 500 p.s.i. although the limiting value was set at 750 p.s.i. The total shear used in the calculation was that which existed at a distance from the support equal to one-half the depth of the member.(34)

Members subjected to direct stress as well as the shearing stress and members with compression reinforcing are expected to behave in accordance with different criteria than those listed above. Methods

(36) F. E. Richart and D. L. S. Larson, "An Investigation of Web Stresses in Reinforced Concrete Beams Part II Restrained Beams"
(34) F. E. Turneaure and E. R. Maurer, Principles of Reinforced Concrete Construction
(35) W. A. Slater, A. R. Lord and R. R. Zipprodt, "Shear Tests of Reinforced Concrete Beams"

[redacted]

such as those proposed by Benscoter and Logan (37) may be adapted to these conditions.

The more important members subjected to this particular condition are the columns of the concrete Buildings No. 3 and No. 5.

2.4.9 Bond Stress in Reinforced Concrete Members

The recent gains in the popularity and the subsequent increasing availability of the high-bond type reinforcing bars provide an excellent and inexpensive means for improving the details of blast resistand structures.

High bond bars were therefore specified for all the reinforced concrete members of the test structure except the footings where normal deformed bars were permitted necause of lower unit stresses in these members and because construction was expedited by the use of an available stock pile of these bars.

The severe requirements for high bond strength in the framing may best be described by the following example. Buildings No. 3 and No. 5 depend mainly on the strength of the bents to resist lateral deflection under the applied loads. These bents consist of continuous girders with columns framing into the girder from above and below, the column steel extending through the girder as in conventional construction.

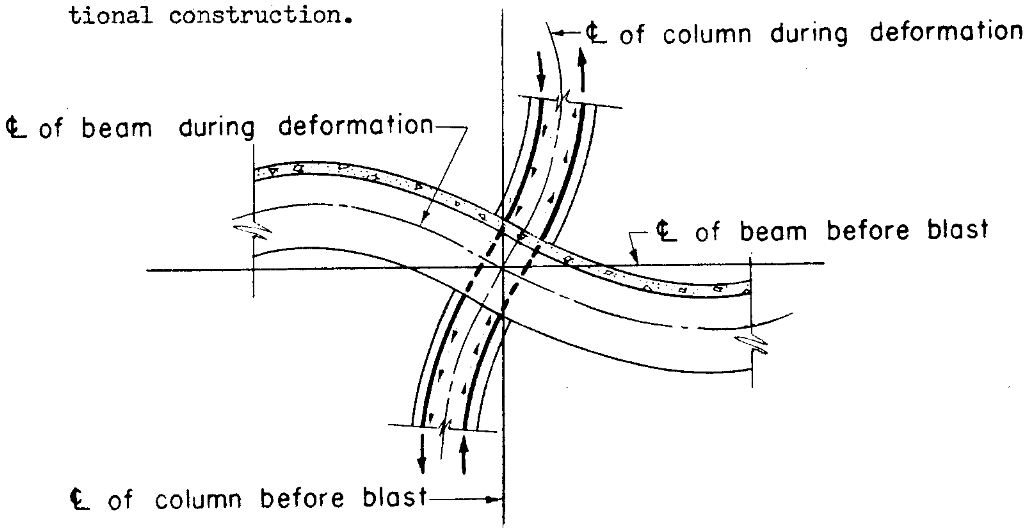

FIG. 2.4.9-1

Under the impact of the lateral load the frame deforms as shown in figure 2.4.9-1. Because of the directions of the bending moments above and below the joint the strain acts to produce compression in the bars on one side of the girder and tension on the

(37) S. U. Benscoter and S. T. Logan, "Shear and Bond Stresses in Reinforced Concrete"

[redacted]

other. This push and pull force on the two opposite ends of the same bar is resisted only by the bond within the depth of the joint. As the steel will reach yield point on each side of the connection, the anchorage must develop double the yield point strength in a relatively short length of the bar. Any slip of the bar in the depth of the girder will act to reduce the moment capacity of the column having the steel on the compression side and will increase the strains of the column on the other side where the same bar is utilized as tension steel. Even if the ultimate strength is not materially reduced by this slip, the strains will be increased, the resisting moments will develop at a slower rate, and an additional variable will be added which will affect the interpretation of the recorded motions.

Tests by the Bureau of Standards (38) indicate that beams with a 20 diameter lapped splice will develop the yield point strength of a normal deformed bar. These tests would indicate that the bars will develop at least 500 p.s.i. According to F. E. Turneaure (34) normal deformed bars will behave as plain bars until the adhesion is broken whence the average unit bond stress will increase with greater amounts of slip. This stress will increase until the concrete splits or the beam fails in other ways. Curves (a) and (b) of figure 2.4.9-2 show the slip required by plain bars and deformed bars respectively to develop given values of bond stress.

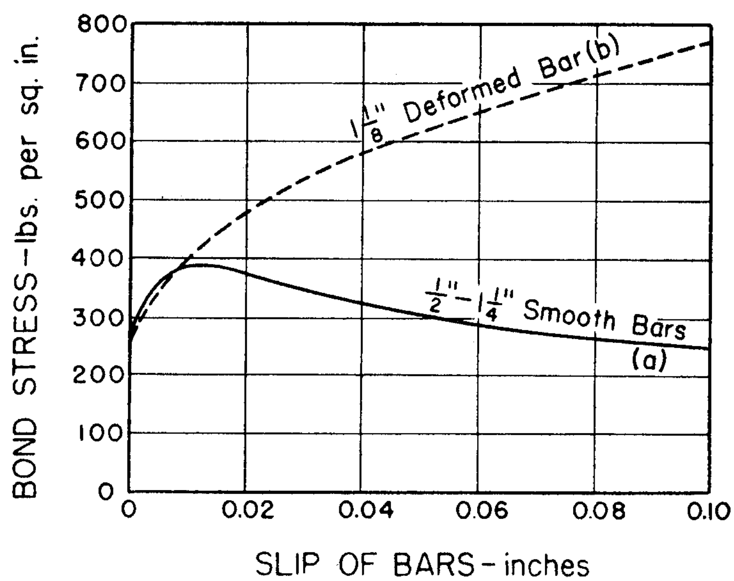

FIG. 2.4.9-2

(38) R. W. Kluge and E. C. Tuma, "Lapped Bar Splices in Concrete Beams"

(34) F. E. Turneaure and E. R. Maurer, Principles of Reinforced Concrete Construction

A more recent series of pull-out tests was published recently by Clark in the American Concrete Institute. (39) These tests were quite exhaustive, describing the resistance to slip of 17 different types of deformed bars. The slip was measured at the loaded and free ends, with three tests being made for each type of bar. These tests proved that the high-bond type of deformation was definitely superior to the bars with the normal deformations. While the test information is not presented in the form desired in order to show the maximum possible load at higher strains, they do show bar stresses of from 40 to 50 k.s.i. and slips less than 1×10^{-3} in. for an 18 diameter embedment of bars cast under the weight of approximately 15 inches of wet concrete. While more test data is necessary, the existing tests indicate fairly conclusively that the necessary bond resistance may be developed by conventional use of bars of high-bond type without reverting to other special and expensive methods of anchorage.

While the bond stress in the other members of the test buildings are also fairly high, they are in general less than those in the column-frame connections, and are similar to the bond stresses in conventional beams loaded to the ultimate capacity.

Rate of Loading

There is little or no data on the effect of dynamic loads on the bond stress, the laboratory beams tested under dynamic loads having relatively low values in both shear and bond. There seems no reason to believe, however, that the high-bond bars, which depend primarily on bearing, shear, and tension in the concrete, will not have strength increases similar to that of the concrete cylinders under the higher rates of loading.

2.4.10 Elastic and Plastic Buckling Stability

The local buckling stability of wide flange and light gage members under static loads is discussed in section 2.4.3-B and C, where numerous references are listed which discuss this subject in detail.

The steel columns of the mill building, the web members of the trusses, and the columns of the steel frame building, which are designed to carry high direct stress plus twist in the framed joints, must also be investigated for overall buckling stability.

For static stresses within the elastic range the procedures discussed in standard text books (13), (15) may be used directly. For dynamic loading, studies published by the National Advisory Committee

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(39) A. P. Clarke, "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars"

(13) S. Timoshenko, Theory of Elastic Stability

(15) J. E. Younger, Mechanics of Aircraft Structures

[redacted]

for Aeronautics (32, 40) give valuable information for particular conditions

As a conservative method for the design of members beyond the elastic limit it is suggested that for any particular small time interval the dynamic character of the loading be disregarded and that the members be designed for axial loads sufficiently below yield point values so that for any given or expected deformation a moment capacity can be developed which will provide for static load equilibrium.

To achieve this condition it is assumed that the axial loads are carried at yield point stresses on the area of the member nearest the neutral axis, as was assumed in calculating the fully plastic bending moment in section 2.4.3-E. The remainder, or outer portions of the member are assumed free to develop the bending moment which provides load equilibrium in the deflected position. However, if the unused portion of the member during any time increment is capable of developing the lateral bending plus the moment due to the load eccentricity and including the eccentricity due to the applied moments, the member may be assumed safe.

If the member is subjected to an axial load great enough to cause compression yield point stress across the entire section, the member will be unstable and will fail unless the load is extremely brief and the column mass is fairly high. This failure condition is consistent with the adopted stress-strain curve which assumed that the stress does not increase for strains beyond the yield point value.

For specific designs it may be necessary to investigate a few of the more critical conditions in further detail. As the moment will probably be variable in the length of the member, possibly the simplest method is to use a numerical procedure. Within the elastic range the method published by Newmark (41) may be applied directly. If the stresses exceed the yield point this solution may be extended as follows:

a. First the axial load is assumed to be carried at yield point stresses in the area nearest the neutral axis. Under the assumption of constant resistance after the stress reaches this yield point value, this area will not contribute in resisting bending.

b. The remainder of the member will contribute resistance against deflection and the effective moment of inertia may be considered as consisting only of this unused portion of the member. The elastic deflection under the variable moment due to lateral loads, the moment due to load eccentricity, and the moment due to deflec-

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(32) C. Koning and J. Taub, "Impact Buckling of Thin Bars in the Elastic Range Hinged at Both Ends"
(40) J. Taub, "Impact Buckling of Thin Bars in the Elastic Range for Any End Condition"
(41) M. N. Newmark, "Numerical Procedure for Computing Deflections, Moment and Buckling Loads"

tion stresses may then be computed by the same method as described above. The same method may be even further extended, if the proper stiffness is used in the various steps, to the condition where the entire section is just less than fully plastic.

Great accuracy is not required in the methods of determining the buckling stability of the main frames as the particular relationship of the relatively short duration of the peak roof loads (producing axial stress) to the high lateral loads (producing large bending moments at the ends of the of the members) results in member sizes more than ample to provide buckling stability.

2.5 Summary of Recommended Design Methods

Contents

2.5.1 Introduction

To summarize the preceding discussions, Sections 2.5.2 to 2.5.3 list in outline form the methods recommended for use in the design of blast resistant structures. The procedures are simplified whereever deemed advisable and are believed, with the exception possibly of the analysis of frames with participating walls, to be entirely suitable for practical use. It may be expected that some of the assumptions will require modification as additional data becomes available; however, it is believed that a substantial portion of this data will be furnished by the proposed test.

2.5.2 General Design Data

A. Increase in Strength with Rate of Loading

The effect of the rate of loading on strength of steel members is summarized in figure 2.5.2-1. The derivation of this curve is described in Section 2.4.2

The time to initiate plastic strain may be obtained either from the natural period, if readily available, or by computing the time required to reach yield point stress using a step-by-step process and a stress-strain curve increased beyond static yield point by an assumed amount which in turn may be confirmed or modified by trial.

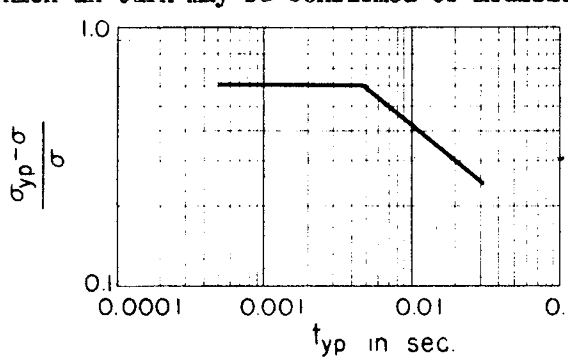


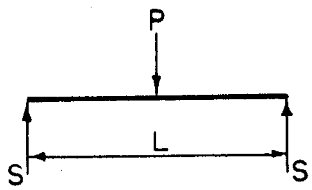
FIG. 2.5.2-1

The effect of rate of load on the strength of reinforced concrete flexural members is included in the suggested resistance function as given in Section 2.5.2-C below.

[redacted]

B. Design Constants for Members under Dynamic Loads

1. Simple Beams or Slabs with Concentrated Loads at Midspan



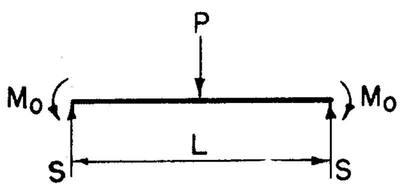
Load Mass Factor = $LMF = \frac{1}{3}$

R = End Shear corresponding to static resistance

S = Total Reaction at any instant

$= R - \frac{1}{2}\left(\frac{P}{2} - R\right)$

2. Fixed End Beam or Slab with Concentrated Loads at Midspan



$LMF = \frac{1}{3}$

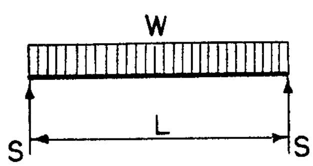
M_o = Moment at the support

M_c = Moment at Midspan

$$R = \frac{2(M_o + M_c)}{L}$$

$$S = R - \frac{1}{2}\left(\frac{P}{2} - R\right)$$

3. Simple Beam or Slab with a Uniform Load

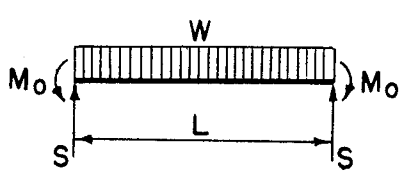


$LMF = \frac{2}{3}$

R = End Shear corresponding to static resistance

$$S = R + \frac{1}{4}\left(\frac{W}{2} - R\right)$$

4. Fixed End Beam or Slab with a Uniform Load



$LMF = \frac{2}{3}$

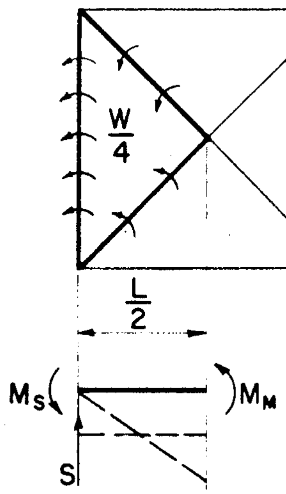
M_o = Moment at the supports

M_c = Moment at Midspan

$$R = \frac{4(M_o + M_c)}{L}$$

$$S = R + \frac{1}{4}\left(\frac{W}{2} - R\right)$$

5. Square Panel under Uniform Load



$LMF = 1/2$

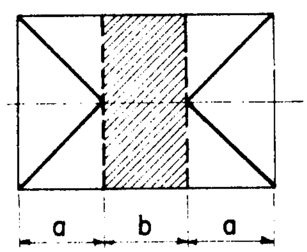
M_S = Total moment along failure plane at support

M_M = Component, perpendicular to the supporting edge, of all the moments along the failure planes of each triangle

$$R = \frac{M_S + M_M}{L/6}$$

$$S = R + \frac{1}{3}\left(\frac{W}{4} - R\right)$$

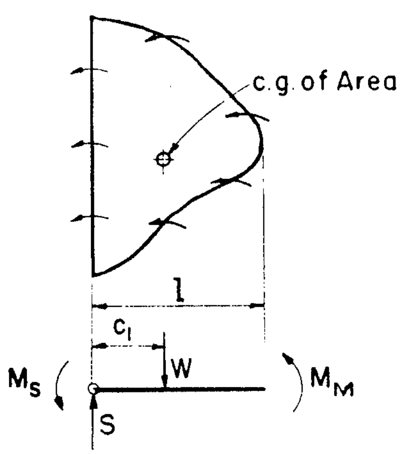
6. Rectangular Panel under Uniform Load



a) Triangular Sections; Same as for square panel (case 5)

b) Same as for one-way slab (case 4)

7. For Any Other Shape under Uniform Load



M_S = Total moment along failure plane at support

M_M = Total component perpendicular to supporting edge, of all moment along the interior failure planes

I_c = Moment of inertia about an axis through center of gravity, parallel to supporting edge

W = Total load on area

$LMF = \rho^2/cl$ where ρ = radius of gyration about supporting edge.

$$R = \frac{M_S + M_M}{c}$$

$$S = R + \frac{1}{1 + \frac{c^2 m}{I_c}}(W - R)$$

C. Resistance Functions

The resistance functions may be obtained by the design methods stated in the following sections:

1. Reinforced Concrete Members

   Bending without axial load, Section 2.4.3 H

   Combined bending and axial load, Section 2.4.3 I

2. Members of Structural Steel

   Bending without axial load, Section 2.4.3 D

   Combined bending and axial load, Section 2.4.3 F

Except for quick acting simple and restrained beams, the resistance function will be of the type shown in figure 2.5.2-2 with some modification if subjected to axial load. The arbitrary limit for the use of this curve will depend on the time required to initiate plastic strain. Figure 2.5.2-2 will be assumed to apply for all cases where the time necessary to reach yield point is greater than 0.01 seconds.

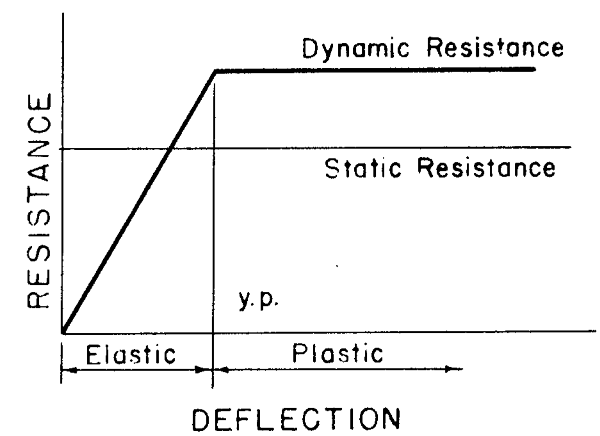


FIG. 2.5.2-2

Members loaded to yield point in less than 0.01 seconds may be assumed to have a resistance function varying with respect to time rather than displacement (see Section 2.4.3). This results in simplifications of the design of the bulk of the component members of the structure. The resistance function

for these members is shown in figure 2.5.2-3.

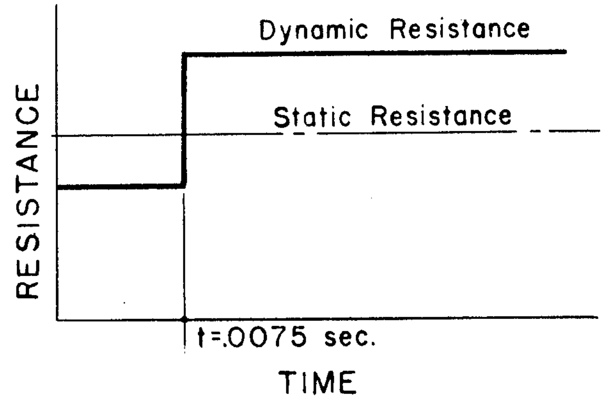


FIG. 2.5.2-3

2.5.3 Design Procedure

A. Curtain Walls and Slabs

Notation: Z_D = Section modulus required to resist the dynamic load

M_D = Moment due to the dynamic load (will be some multiple of the static moment)

M_s = Calculated moment using the peak dynamic load as a static load

σ_s = Permissible stress under static load

σ_d = Permissible stress under the dynamic load
= $C \cdot \sigma_s$ where C , may be obtained from figure 2.5.2-1

T = Natural period of vibration of the member

t = Time during which the member is deflecting

1. One-Way Members

a. Light weight brittle members

$$Z_D = \frac{M_D}{\sigma_d} = \frac{2\,M_s}{\sigma_d}$$

the required Section Modulus being less than twice that required for a static load of the same intensity because the value of σ_d will be greater than σ_s .

[redacted]

b. Light Weight Ductile Covering (allowing a maximum rotation within any plastic hinge of 0.125 radians)

$$Z_D = \frac{M_D}{\sigma_d} = \frac{1.10 M_S}{\sigma_d}$$

However, Z_D shall not be less than required by $\frac{M_S}{\sigma_S}$ if the pulse is essentially rectangular and of load duration Δt greater than $3T$.

c. Heavy Ductile Members (allowing maximum rotation within any plastic hinge of 0.125 radians)

Case I: For a pulse with a zero time of rise and a constant or near constant value thereafter:

$$Z = \frac{M_D}{\sigma_d} = \frac{1.10 M_S}{\sigma_d} \quad \text{if } \frac{T}{4} < t < 3T$$

However, if the duration of the load is longer than the expected duration of the increase of strength due to rate of loading, assumed at $3T$. the design section modulus shall not be less than required to carry the load at the permissible stress under static load, or $Z = M_S/\sigma_S$.

Case II: If the applied load varies with respect to time and the resistance function is known with respect to time as shown in figure 2.5.2-3:

a. Assume a member size (the methods of Case I may be used for the first trial) and estimate its static strength.

b. Compute the natural period of the assumed member.

c. Estimate the expected increase in strength using figure 2.5.2-1. Establish the resistance function with respect to time. (See section 2.5.2 C.)

d. Select time stations $t_0 \ldots\ldots\ldots t_f$, such that each time interval $t_f - t_{f-1}$ is a constant time interval. Record in column (1) of the table in figure 2.5.3-1

e. From the given loading curve obtain the average unit load pressure P acting during the time intervals $(t_1 - t_0)\ldots\ldots(t_f - t_{f-1})$. Convert these unit pressures to total loads P and record in column 2.

[redacted]

f. From the resistance function obtained by step (c) find the average total member bending resistance for each time interval. Record this value in column 3.

g. Add the net average impulse $P-R$ for each time interval to the total impulse of the preceding time station thus providing the total impulse of each time $t_0 \ldots\ldots t_f$. Record in column 4.

h. Sum the total impulses of column 4 thus obtaining the summation $\left(\Sigma\left[\Sigma(P-R)\right]\right)$.

i. Multiply the total mass by the load-mass factor to obtain the equivalent mass (m_e). The load-mass factor (L.M.F.) may be obtained from the table of constants listed in section 2.5.2 B.

j. Multiply the summation of the impulses $(\Sigma[\Sigma(P-R)])$ by the necessary constants thus obtaining the total deflection. If the deflection is unsatisfactory choose a new beam size and repeat steps (a) to (j).

1	2	3	4
t	P	R	ΣI $\Sigma(P-R)$
t_o			0
	—	—	
t_1			—
	—	—	
t_2			—
t_{f-1}			—
	—	—	
t_f [1.]			0
	(5) $\Sigma[\Sigma P-R]$ =		—

$$\delta = \frac{\Sigma\left[\Sigma P-R\right]\Delta t^2}{m_{eff.}}$$

1. Analysis complete at t_f

FIG. 2.5.3-1

[redacted]

Case III: For slower acting members in which the resistance function with respect to deflection rather than with respect to time must be used, prepare a table similar to the table in figure 2.5.3-1 of the preceding example but extend it to include tabular columns giving the actual deformation for each time interval. Compute deformations as outlined below and as illustrated by the table in figure 2.5.3-2.

Then

a. Choose a convenient time for each time station. Compute the time interval Δt between each time station. Record in columns 1 and 2 of the table in figure 2.5.3-2.

b. Record the average applied pressure P for each time interval in column 3.

c. Obtain the load-mass factor from Section 2.5.2 B. Use the same mass factor throughout.

d. Within the elastic range:

   1. Assume a resistance for the first time interval.

   2. Calculate the deflection during the first time interval using the resistance assumed in step 1.

   3. Check the calculated deflection against the resistance-displacement curve. If the assumed resistance does not agree with the actual resistance for the calculated deflection, pick a new resistance and repeat steps 1 and 2.

   4. When the calculated deflection checks the trial resistance, record R and advance to the next time increment. The trials will converge quickly.

e. In the plastic range the resistance is known and is constant for each step.

The computations may be simplified by adding column 12 to the table in figure 2.5.3-2. Then the R value for deflections within the elastic range may be computed by using columns 1 to 11 as described in steps (d-1) to (d-4). Steps 5 to 11 are not needed (as the value of R is known and constant for all steps) beyond the elastic range. Column 12 is then filled and summed similar to Case II.

[redacted]

1	2	3	4	5	6	7	8	9	10	11	12
t	Δt	P	R	$\overline{F}$	a	Δv	Σv	$\overline{v}$	Δx	x	$\Sigma(P-R)$
t_0	—	—	—	—	—	—	0	—	—	0	0
t_1	—	—	—	—	—	—	—	—	—	—	—
t_2							—			—	—
.							.				.
t_p[1]	—	—	—				—				—
.											.
t_{f-1}	—	—	—								—
t_f[2]											0
									$\Sigma[\Sigma(P-R)]$		—
Chosen Time Stations	Time Increments	P From Given Load	R From Resistance Function	$F = P - R =$ Net Unbalanced Force	$a = \frac{\Delta v}{\Delta t} = \frac{\overline{F}}{m}$	Change In Velocity During Time Interval $= a \cdot \Delta t$	$v_n = v_{n-1} + \Delta v$	$\overline{v} = \frac{v_{n-1} + v_n}{2}$	$\Delta x = \overline{v}\Delta t$	$x_n = x_{n-1} + \Delta x$	

1 Plastic State Reached At t_p
2 Analysis Complete At t_f

FIG. 2.5.3-2

2. Slabs

a. Square Panels

The analysis of the square panels is identical to the analysis of the one-way members as described above, except that the loading constants and the load-mass factor will be as noted in Section 2.5.2.

[redacted]

b. Rectangular Panels

Rectangular panels may be considered as being composed of both triangular and one-way slabs as shown in figure 2.5.3-3. The end portions ABCD and A',B',C',D' are analyzed as triangular panels in the same manner as described above for square panels. The center portion B,D,B',D' may be analyzed as a rectangular, one way slab.

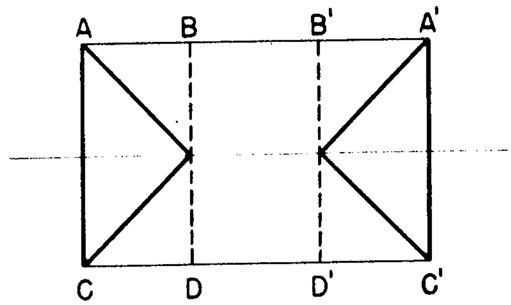


FIG. 2.5.3-3

B. Frames

1. Frame Action Without Wall Participation

a. Find the static resistance function for the columns under varying axial loads as a function of deflection. Sketch a curve through calculated points thus providing a continuous resisting function.

b. Estimate the increase in yield point due to dynamic loading (use figure 2.5.2-1).

c. Compute and tabulate the axial load on the columns for each time interval $t_n - t_{n-1}$ during the expected time of motion.

d. Compute and tabulate in the table in figure 2.5.3-4 the average translational blast forces for a sufficient number of suitable time increments $t_n - t_{n-1}$ to cover the expected duration of motion.

e. The analysis of the frame motion will then proceed similar to the analysis of heavy, ductile members, with the following exceptions:

(1) While the motion of each floor is analyzed separately, the analysis of all floors proceeds simultaneously, and the relative deflections of the floors are tabulated at the end of each time increment.

(2) The net resistance to each floor is the algebraic sum of the column shears acting above and below the floor. The shears are, of course, determined by the column moments which exist during the particular time interval, the moments in turn being a function of the axial load and the relative deflections of the floors at that particular time. Comparative studies show that the redistribution of moments and shears due to elastic frame action can be ignored completely without introducing sensible errors except in the computation of column loads and moments on the foundations.

f. The design procedure for each floor level is illustrated in the table in figure 2.5.3-4. A sufficient number of trials must be made for each time interval so that the estimated value of R given in column 14 agrees with the computed value given in column 13 of the table.

g. During the late stages of the forward motion, some columns re-enter the elastic range and in some cases the moments reverse. Provision for this reversal must be made in the determination of the resistance function of the columns.

2. Frame Action Including Wall Participation

The analysis of frames in which the exterior walls participate in the frame action, while entirely possible as used in the design of the test structures, is not yet sufficiently simplified to be considered as a practical design method. In general, the analysis proceeds in the same manner as outlined for multi-story frames without wall participation except that the distribution of the lateral loads to the various floor levels is dependent on the configuration of the continuous front and rear wall panels, which is determined by the relative deflections of the building as a whole. Hence both the wall shears and the column shears are functions of the relative deflections of the various floors, and both must be determined by successive trials. The analysis of the wall panels at the end of each trial is the only modification necessary to the procedure outlined above for the analysis of frame action without participation of the walls. A detailed description of the analysis is made in Section 2.4.5.C-4 and a sample calculation is shown in Appendix A-5.

[redacted]

15	$\bar{F}$		Load (Known)
14	$\bar{R}_{ass'd}$		Assumed Average Resistance
13	$\bar{R}_{calc.}$		$\bar{R} = \bar{r} - \bar{r}_a$ = Total Average Resistance During Δt
12	$\bar{r}_a$		Calculated Average Resistance Of Columns Supporting Floor Above
11	$\bar{r}$		$\bar{r} = \frac{r_{n-1} + r_n}{2}$
10	r_n	0 . . .	Calculated Resistance At t_n Of Columns Supporting This Floor
9	$\Sigma \Delta x_b - \Delta x$		Total Deflection Of This Floor Relative To Floor Below
8	$\Delta x_b - \Delta x$		Deflection Of This Floor Relative To Floor Below
7	Δx_b		Deflection Of Floor Below During Δt
6	x	0 . . .	$x_n = x_{n-1} + \Delta x$
5	Δx		$\Delta x = \bar{v} \cdot \frac{m}{\Delta t} \quad \frac{\Delta t^2}{m}$
4	$\bar{v} \cdot \frac{m}{\Delta t}$		$\bar{v} \cdot \frac{m}{\Delta t} = \frac{v_{n-1} + v_n}{2} \left(\frac{m}{\Delta t}\right)$
3	$v_n \cdot \frac{m}{\Delta t}$	0 . . .	$\Sigma \bar{F} - \bar{R} = \Sigma \Delta v \cdot \frac{m}{\Delta t} = v_n \cdot \frac{m}{\Delta t}$
2	$\bar{F} + \bar{R}$		Net Translational Force During Δt
1	t	t_0 t_1 . . . t_{f-1} t_f	Chosen Time Stations

1. Analysis completed at t_f.

FIG. 2.5.3-4

[redacted]

PART III. INSTRUMENTATION AND SURVEY OF THE BUILDINGS

Contents

3.1 General

Under even the most favorable circumstances, observations dealing merely with the survival, or the non-survival of the various test structures will give little information concerning design procedures and characteristic behavior of materials under dynamic loads unless the characteristics and intensities of the applied loads are known in detail. Attempts to compare the degrees of damage suffered by the various types of buildings may be little more helpful, since the different buildings, having different modes of failure depending on the relative sensitivity of each type to different intensities and impulses, have no common scale on which the various types of buildings can have their respective damages properly rated.

Furthermore, information is needed to show the modifications necessary in the design procedures so that any building may be designed to resist any specified blast loading.

In view of this it would be unwise to conduct the intended tests without devoting a respectable fraction of the total funds to a scientifically worked out program of instrumentation carefully chosen to provide the necessary basic information.

In order to judge the minimum number and types of records that would be most suitable, various methods for utilizing measurements in the intended post-test analysis were studied and based on these studies a recommendation for minimum instrumentation was included in the preliminary report. This report was then discussed at a meeting with the contracting authority and all of the special consultants.

After the decision was made by the contracting authority to have the supply, installation, and operation of the instruments handled by a special separate group, a further meeting was held in the contractor's

[redacted]

office at which time a representative of the contracting authority, Dr. R. J. Hansen, who formulated the final instrumentation layout, and representatives of the contractor restudied the question of the instrumentation, expanding the scope as permitted by the expected allocation of available funds. Drawings showing the location, type, and number of instruments were then prepared by a separate contractor.

Under the circumstances it seems best to describe the final instrumentation layout merely by including certain of the above described drawings in a later section of this report. (See Appendix 4.)

On the basis of the several given estimated pressure loadings, this contractor furnished computed values for the expected accelerations, velocities, and displacements of the various buildings, frames, and structural members to help judge the estimated range required for each of the numerous instruments. Tabulated estimates of these maximum pressures, accelerations, velocities, and displacements which the various gages may have to measure will also be included in a later section. (See Appendix 4.)

In anticipation of a complete report by the instrumentation contractor, the instrumentation will be discussed only in a general way in this report. This planned instrumentation layout in addition to the survey of the building before and after the test should meet all reasonable requirements for test records even after making liberal allowances for inevitable percentage of instrument failures and for possible losses of records from collapses of the structures. The instrumentation program seems adequate and well balanced.

3.2 Types of Measurements

The design procedures in the case of buildings resistant to A-bomb blast must be based essentially upon the dynamic behaviors of the structures under large and rapidly varying loads. For any reasonably complete knowledge of dynamic behavior, a number of continuous timed records are essential.

Theoretically, the quantity measured may be velocity, acceleration or displacement since the time-velocity curve, and the time-acceleration curve follow directly from the time-displacement curve as successive derivatives and one curve can be had from any of the others. It therefore is theoretically immaterial whether the selection is acceleration-meters, velocity-meters, or displacement-meters and the decision in different cases may be on the basis of convenience or other practical considerations. Velocity-meters have apparently little to recommend them. Displacement-meters capable of measuring absolute displacements of more than a couple of inches may be difficult to design.

It also does not seem too important whether absolute measurements (referred to space) rather than relative measurements (referred to some other point in the moving structure) are selected. However as the error

[redacted]

in an absolute measurement is unaffected by the error in any other measurement, that, in itself, is an advantage. But where interest is primarily centered in the inter-story horizontal displacement, as in studying the horizontal inter-story resistances in a frame structure, it may be simpler and more accurate to measure the relative displacements directly rather than to deal with the difference between measurements of the absolute displacements of the two adjacent stories.

As either type of measurement offers its own special advantages and disadvantages in the individual case, the choice of absolute vs. relative measurements is best settled in each case by judgement.

3.3 Types of Instruments

The various types of instruments usable for this test are perhaps most clearly and briefly classified according to the ways in which they combine four attributes from the four groups in table (3.3).

Table 3.3

GROUP	ATTRIBUTE
1. variable measured	1.1 blast pressure
	1.2 acceleration
	1.3 velocity
	1.4 displacement
2. character of measurement	2.1 absolute value
	2.2 relative value
3. type of record	3.1 continuous record
	3.2 indication of maximum
4. method of recording	4.1 self recording
	4.2 distant recording

It has been previously explained and emphasized that, since dynamic behavior is being analyzed, it is essential to have a number of measurements as continuous functions of time. Self recording instruments are cheaper and handier. Their magnetic tape records are perhaps not very readily damaged, and the chances of catastrophic collapse of structures in which they are placed are probably extremely small. Nevertheless, to insure positively that a good number of continuous records up to the time collapse will survive in the worst case, self-recording instruments are not exclusively relied on. Both self-recording and distantly recording instruments are represented in different proportions in the intended layout.

Caution should also be exercised to avoid omitting other types of measurements merely because continuous measurements are the best and

only kind which can lead to ideal solutions. There is, however, often a surprising amount of information to be had from a knowledge of the maximum relative displacement alone.

Where there is no permanent relative displacement, a gage of some sort is needed to indicate what the maximum relative displacement has been. Where there is relative permanent displacement (measurable by surveying the structure before and after the test) it is necessary only to correct permanent displacement for elastic rebound in order to have maximum displacement.

The only exception is where relative motion has reversed its direction after maximum relative displacement has occurred. But aside from those cases where it is known, or even only suspected that this has occured, there will remain large numbers of cases where survey measurements can be used with confidence.

The self recorded instrumentation will therefore be amplified by surveys of the buildings before and after the test. Locations of permanent bench marks on the test structures are shown by the contractor on a special drawing which has been sent to the builder of the test structures; this drawing is shown on Sheet 13, Appendix 7 of this report. To decrease the probable errors in recording the permanent displacements determined in this way, the measurements should be checked both before and after this test.

Besides the instruments shown in Appendix 4, numerous auxiliary stress-strain gages and gage points will be provided in the steel beams and columns and on the reinforcing steel of the concrete beams and columns.

Most of the instruments will be located on the heavy concrete panels, the frames, and the shear walls. The thin coverings on the end panels are so numerous and so difficult to instrument that these members were provided through a range of strength so as to be self recording. It is necessary only to know the applied pressures, the type of failure, and the maximum girt spacing on which the coverings did not fail.

Surfaces of all concrete, and of all structural steel members, should be whitewashed to make cracks in the concrete more visible, to indicate plastic flow in the steel, and to improve photography. Photography should be exceedingly complete because it may be the only way of clearing up doubtful points of which the full importance is not immediately realized.

As described earlier, the live load is simulated by means of sand bags stacked in a more or less evenly distributed fashion on the floor. These sand bags are restrained against lateral motion by timber bulkheads placed on 4 ft. centers in each direction. A method of instrumentation should be devised whereby any motions of this load with respect to the floor will be known for the post-test analysis.

In addition to the above information, cylinders and test beams are required by the Technical specifications for contract W49-129-eng-148. These specimens will provide samples of actual construction materials for later static and dynamic tests.

[redacted]

3.4 Summary of Desired Information

A - Pressures:

Time-pressure curves are needed in a sufficient number of places to give:

1. A complete pattern of pressures on the exterior sides of the front, rear, and end walls.
2. A complete coverage of the roof surface including the pressures reasonably near the front edge.
3. Pressures on the inner walls of the building with windows.
4. Pressure on the footing projecting beyond the front wall.
5. Some interior pressures to check the net pressure acting on the wall and roof panels.
6. The pressures in the entranceway and doors of the personnel shelter.
7. The earth pressures beneath the footings of the main test structure and around the sides and circumference of the personnel shelter.

B - Continuous Records:

1. Continuous records of accelerations, velocities or displacements should be provided in sufficient numbers to check or corroborate the expected action of all types of one-way members.
2. The same type of readings should be provided to define the relative horizontal and vertical displacements of the various floors of the different buildings.
3. A continuous record should be provided which will show the behavior of the live load with respect to the floor.
4. If possible, continuous strain measurements should be obtained which will show the axial load on the various columns.

C - Permanent Records:

1. A complete system of bench marks should be established so that the permanent floor and panel displacements may be accurately established.
2. Numerous closely spaced gage points should be provided adjacent to the connections of the beams and columns to show the unit strain and the total length of the plastic hinges.

3. Strain points should be established if possible which will indicate the slip in the connections and the rotation of the column bases.

D - Photographs, Drawings and Descriptions:

Photographs, drawings and written discussion should describe every type of behavior which can be judged by the visual post-test examination of the structure. This information should include the general patterns of behavior and should note indications of local bond, shear, or bending failure.

E. Test Specimens:

1. Sufficient test specimens should be taken to provide an accurate estimate of the actual static strength of the materials used in the test buildings.

2. Additional test specimens should be provided so that later dynamic tests may be undertaken in the laboratory where necessary to substantiate points not otherwise covered in the test structure.

3.5 Form in Which Instrument Data Should be Presented

The primary purpose of the measurements which will be recorded on this test is to make the data conveniently available for use in certain dynamic problems of structural engineering, and the report of the instrument group should be prepared with this in mind.

The personnel making the post-test analysis may not be as well prepared as the group doing the instrumentation to do the work of fitting individual points, eliminating instrumental errors and irregularities, and integrating and differentiating these curves by various methods which, in the hands of different individuals, would probably give minor but annoying discrepancies.

It is therefore recommended that the report of the instrument group include (a) time-pressure curves smoothed only as necessary to eliminate instrumental errors and irregularity from all the blast records, (b) similarly smoothed time-acceleration, time-velocity and time-displacement curves; i.e. all three of these curves regardless of whether only displacement-meters or acceleration-meters were used.

PART IV. RECOMMENDED PROCEDURE FOR THE ANALYSIS OF FIELD MEASUREMENTS

While every possible care was exercised to use past experience and available test data as the criteria for the design of the test structures, numerous questions still arose which could not be definitely answered on the basis of available information. Some of these were necessarily settled in a somewhat arbitrary manner, while others are subject to the uncertainties involved in extrapolating data from static to dynamic conditions and in considering strains beyond the elastic range.

These uncertainties, as discussed in Part II, concern the loading on the various parts; the dynamic behavior of the materials; the basic assumptions in the design; and the appropriateness of the simplifications used in the design analysis.

It is believed that the major errors are likely to occur in the predicted pressure loadings as so many factors can contribute to error both in the estimate of the explosive equivalent of the bomb and in the complicated analysis of the shock wave effect. While the information provided to the contractor is based on a studied analysis by leading authorities and represents the best estimate possible; it must be emphasized that relatively minor differences, percentagewise, between the actual and predicted forces, whether in intensity, duration or distribution, will have a major effect on the behavior of the test structures.

It is necessary, therefore, that a careful study be made of the entire question of applied pressures. This means an examination of the recorded pressure readings, including a consideration of the sensitivity and accuracy of the instruments, and a synthesis of new loading curves similar in form to those originally furnished.

It would seem that this study can best be made by a cooperative effort on the part of the group furnishing the initial theoretical pressure curves and the contractor specializing in the instrumentation. The reconstructed pressure loadings obtained from the analysis must be correct, otherwise their use on the various structures would result in erroneous modifications of the proposed design procedures.

It would be preferable if this data could be furnished immediately after the completion of the test (and without unnecessary delay for completing those other parts of the instrument group's report which cannot be used until after the blast pressure data is used).

When blast-pressures are known, all calculations relating to behavior of the structures must be repeated, using the newly derived values for blast pressures and the actual strengths of the material. Only after this is done can the post-test analysis be properly appraised.

The personnel who made the original computations for the design of these structures made repeated revisions in the test buildings to conform with revised estimates of blast pressure. It is believed that this same group would not find this additional recomputation too lengthy a process for the single specific set of loads found on the test structures. It is also believed this work could proceed, while the instrument group was completing the rest of its report.

As previously mentioned, the group in charge of the instrumentation would seem to be in the best position to assess the accuracy of the instruments, to correct the irregularities in the data, and to reduce the recorded data to curves representing the accelerations, velocities and displacements of the individual points. This information, as well as the original data, should be available for study and interpretation by the persons making the post-test analysis.

The building contractor must furnish, under Articles 2-05(1), 2-05(2), and 5-05 of the Technical Specifications, numerous test specimens of the actual construction materials. Certain of these specimens are designated for static tests which will provide basic design information on the strength of the members. Reports of these tests should be furnished with or prior to the submittal of the revised blast pressures.

The remainder of the specimens are to be set aside for dynamic tests in case corroborative information is needed beyond that provided by the test of the prototype building.

Having the actual pressure loading as recorded on the test structure, the static strength of the members from the test of the specimens, and the recorded characteristics of accelerations, velocities, and displacement obtained from the instrument recordings; a detailed re-analysis of the buildings should be undertaken. By comparison between the corrected theoretical behavior and the actual recorded behavior, the major and minor assumptions may then be studied in detail.

It is difficult to judge the manner in which the predicted and actual behaviors will differ and as a result the detailed analytical approach must await the test results. It is believed, however, that the assumptions described in Part 2 are fundamentally sound and, barring large discrepancies in the loading, the modification to the design methods will involve only a quantitative appraisal and modification of the empirical design factors. In this case the summary of the anticipated detailed approach to the re-analysis may be described as follows:

> The features distinguishing the design of blast resistant structures from conventional design are:-
>
> (a) The use of solutions based upon dynamic relationships.
>
> (b) The realistic assumption that a building will not "fail" when the elastic limit is reached and that it will still remain serviceable after a major plastic deformation has taken place.

In general the solutions involve semi-empirical treatments. These derivations involve equations which, in form, are as consistent as practicable with the known, or assumed known, characteristic behavior, but

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which have, quantitatively, various areas of uncertainty represented by estimated parameters requiring experimental evaluation. The measurements to be made in the test are for the purpose of establishing the most suitable values of these tentatively estimated quantities.

Individual cases differ, but there are, in general, two principal ways in which a constant in a semi-empirical formula may be determined from experimental results. By method (a) we may reverse the direction of normal computation with the formula, such as the formulas and step-by-step procedures of Part 2, so as to work backwards to derive a corrected value for the constant as a direct function of the measured variable and the given quantities.

By method (b) we may use the formula in the normal manner to calculate the measured variable as a direct function of the given quantities and, by varying successively the values of the constant, finally approximate the value which makes the formula correctly estimate the measured variable. Sometimes one, and sometimes the other of these methods is the better, dependent on the success of the final form in satisfying numerous general and varying problems.

The constant is not necessarily satisfactory if based on a measurement at only one point (e.g. at maximum displacement), and, for general solutions, such as are developed in Part 2, will only be so if the same constant proves adequate for all different types of materials, for all types of intensity, duration, and characteristics of loading, and for an appreciable range of deflections. If a continuous test record is provided the best value of a derived constant, for this reason, is the one which will describe accurate values corresponding to all points along the entire record of the continuously measured variable or, what is much the same thing, it is that value of the constant which as closely as possible indicates the entire record of the measured variable.

In some cases where the mechanism involved proves less simple than assumed, no single value or constant may hold very well over the entire range. Some linear, or other simple function may then have to be replaced by a constant; but this should present no great difficulty if sufficient data is available to provide a general answer. One quasi-constant of this nature may, for example, be the ratio between the "effective" mass and the total mass of a beam or panel, because this ratio changes slightly during the beam's deflection.

The time-resistance function (see figure 2.4.3-17) appears to be basic in the analysis of panels, and the displacement-resistance function (see figure 2.4.3-19) seems basic in the analysis of framed structures. Neither of these is perfectly known. They need to be established from measurements recorded during the intended test.

In view of the above comments, the first attempt, probably, will be to derive by reverse calculations following method (a) a series of resistance values corresponding to the measured time and/or displacements. It is to be hoped such points will arrange themselves without much scatter and that they can be fitted by very reasonable resistance curves.

New analyses of the test structure, or parts of the test structure should then be made as required using the new resistance function to recompute new theoretical motions. If these new resistance functions so derived give good agreement with the acceleration, velocity, and displacement curves prepared from the records of the test (which were, for this purpose, recommended in 3.5) it may be assumed (at least for these various panels, members, and the three-story framed test structures) that the proposed methods develop a rational design procedure which works properly. If the plotted points to which the resistance curves are fitted, lie in quite narrow bands, then any significantly different resistance functions probably would not serve nearly so well.

It may be, however, that results will be less fortunate. The plotted points to which the resistance curves must be fitted may be scattered over large areas through which any one of a large number of possible resistance curves may be drawn which will fit the points equally well.

In such case it probably will then prove better to abandon method (a) and to shift to method (b); providing the records, photographs and descriptions do not locate local failures and instrumental errors which might have been the cause of the scatter of the points of the test data. If the scatter still persists, successive reasonable-looking resistance functions may be tried until one is found which approximates a majority of the actual records.

The locations, numbers, and types of instruments to use were decided by judging which measurements should be most useful when put to their intended uses. This included both the number of readings required to cover all desired information and sufficient repetition of similar readings to assure corroboration of results and to account for local failures and the loss of certain information.

There may be cases(e.g. with continuous slabs and panels) where the pattern of cracking, as evidenced in the photographs of the whitewashed surfaces, will give important clues which will clear up doubtful points about the exact modes of failure and so point the way to improved analytical treatments. The appearances of the failures for the same reason, should therefore be carefully scrutinized. From the discussion of Part 2 it will be realized that the recommended method and attack are primarily ultimate design procedures. For the members of the test structure that act in or near the elastic range there is expected to be some deviation between the recorded and predicted measurements. It is believed that the photographs and possibly drawings and description will be of particular aid in the study of these more lightly stressed members.

Photographs, descriptions, and drawings are expected to be useful in numerous other cases. So many different situations arise in the detailed design that these can hardly be included in a report of reasonable length, however, it may be well to mention, as examples, such particular design problems as the reinforcing steel bond in the joints between the girders and columns of the reinforced concrete buildings; or the effectiveness of shear reinforcement in the walls; or the bond and shear characteristics of different depth slabs; etc. In these cases, the photographs may indicate local failures which will show why resistance curves substantiated in some portions of the building may not, apparently, be developed in other parts.

[redacted]

It is recognized that numerous problems in the instrumentation will also occur. One such case, for example, concerns the displacement readings of the wall panels. Here large floor displacements are added to the wall displacements thus increasing the range and reducing the relative accuracy of the wall readings. Because of the presence of such difficulties considerable judgment will be required in many portions of the analysis. This judgment will be considerably assisted by the surveys, the photographs, and the available descriptions and drawings.

The thin coverings are too numerous to permit anything like complete instrumentation. However, as the blast pressures will be known and numerous degrees of strength are provided, it is only necessary to measure the deflection. Where they fail, a study can be made of the character of the damage and, where practicable, the mode of failure may be noted. The deflections and largest girt spacing on which the thin covering does not fail should be recorded.

One important phase of the re-analysis will concern the determination of the effective mass of the structure. To this end it will be necessary to examine the records of the motion of the simulated liveload in order to determine when and how the inertia forces from this source are applied. Any motions or shifting of this liveload adds another variable which must be weighed and adjusted before the resisting functions can be evaluated.

The recommended sequence of procedure for the analysis of the field measurements may then be summarized as follows:

(1) Immediately after the test:

  (a) The blast pressure records should be studied by the instrumentation contractor and, if possible, the personnel furnishing the original pressure curves.

  (b) The personnel doing the structural analysis could simultaneously start on the analysis of the strength of the materials, the stress-strain data, and a study of the photographs and available descriptions.

(2) A study and evaluation of the instrumentation records should proceed in which the acceleration, velocity, and displacement curves should be developed. The structural group, at the same time could start recomputing the theoretical motions under the revised blast pressure loading.

(3) A detailed comparison should then be made between the revised theoretical behavior and the observed motions and the theoretical procedures should be modified accordingly.

(4) The modified procedures should be verified by a new set of calculations using the revised pressure loadings, the actual member strengths, and the modified design procedures.

[redacted]

(5) The design procedures should be simplified on the basis of the final theoretical and experimental information. If the opportunity exists, the studies could be profitably extended to involve buildings of various sizes and proportions under impulsive type loadings other than those of the test structures so that the procedures and range will have general application. It is believed, for example, that the methods presented in this report will also furnish a better solution to the problem of earthquake resistance than any other method so far developed.

[redacted]

PART V. SUMMARY OF RECOMMENDED DESIGN PROCEDURES AND FRAMING METHODS

5.1 Design Methods and Procedures

Under certain conditions particular types of members may be readily and economically designed using equivalent static loads in place of the actual dynamic pressures. These conditions depend on the impulse being essentially constant in value during the permitted deflections. As the duration of the blast is relatively long, a large number of small mass quick acting members, such as corrugated metal, asbestos cement boards, and thin concrete slabs, qualify for this treatment, their total deflection occurring before an appreciable change takes place in the intensity of the load.

The high-mass ductile members, such as heavy reinforced concrete sections and building frames, are much slower in action, the deflections continuing for an appreciable part of the total load duration. These members can be designed to take advantage of the drop in pressure with time, particularly if the members are resisting reflected pressures which have a sharp diedown in intensity within 0.03 seconds after the first shock. The use of equivalent static loads for the analysis of these members does not appear to offer any advantages in theory or accuracy and either a systematized step-by-step solution for each individual case or, if the panels are numerous, use of design curves which pass through a few calculated points is recommended.

The step-by-step solution may be quickly solved for each case regardless of the shape of the resistance function but the solution can be simplified if the resistance is constant after the stress reaches its yield point. As described in part 2.4.3, the solution may be still further improved if the resistance is a function of time rather than displacement.

The resistance function varies with the continually changing direct stress and was so considered in the analysis of the test structures. It is expected that as a practical design measure a single constant value can be assumed for this direct stress, this value being some mean between the minimum and maximum expected. As the pressure variation causing the axial load is still uncertain, it is believed that recommendations for this single direct stress assumption should not be made until the recorded pressures and their effect are studied.

It is believed that the procedures used to design the test structures could be applied to design conventional commercial buildings of minimum strength for fees comparable to those accepted for standard designs.

5.2 Recommendations for Structural Framing

The test structures are not expected to include every possible type of framing. These tests do, however, embrace a large number of framing types and the same methods used to design these particular

[redacted]

structures, may be used with reasonable assurance for the design of other types of framing. Once the methods are proved, exhaustive studies could be made for any of the many coverings, framings, and pressure loadings.

While many materials and framing systems may have sufficient strength if properly designed, a scale of comparative costs must be developed for structures designed to resist the blast load conditions. Some materials competitively favorable for conventional loading are entirely uneconomical for the blast intensities. As a matter of interest the estimated unit costs for the different buildings of the test structures are shown in the table of figure 5.2-1.

Bldg.	Structural Framing	Type of Covering	Cost Per Sq. Ft.
1	Concrete Cellular	Misc. - Concrete & Brick	$12.77
2	Steel Frame	Corrugated Metal with Steel Girts	14.17
3	Concrete Frame	12" Monolithic Concrete Wall	6.45
4	Shear Wall	12" Monolithic Concrete Wall	7.34
5	Concrete Frame	12" Monolithic Concrete Wall	5.82
6	Steel Frame	Corrugated Metal Siding with Steel Girts	13.22
7	Concrete Cellular	Misc. - Concrete, Corrugated Steel & Asbestos Cement	14.59

Figure 5.2-1

While some recommendations on framing methods are discussed below, these suggestions must be regarded as tentative, presuming further elaboration after a complete study has been made of the recorded test data.

A. Recommendations for Curtain Walls or Wall Covering

Fragile, lightweight coverings are believed of value only where the framing must be kept light. These panels will transmit a minimum impulse to the structural frames and the fragments of the panel will be such as to cause a minimum damage on disintegration. The frames supporting this covering must, however, be designed for the portion of the blast impulse carried by the panel before its failure. As shown by past records, the frames and contents of buildings of this type will be exposed to damage from fire and wind after failure of the walls.

Light weight ductile materials, such as corrugated metal, possess greater resilience and will resist higher dynamic loads per unit of static strength than the fragile types. The strength of commercial sizes of this material is substantially lower, however, than needed to resist intense blast pressures unless expensive and impractical systems of support are provided. Comparative cost studies show that solid reinforced concrete walls, which would obviate the necessity of further covering, are cheaper than the steel framing required to support these lightweight coverings.

Heavy monolithic reinforced concrete walls offer the greatest promise as blast resistant members. These walls are not too expensive, can be treated architecturally, and, because of their high mass and the ductility of the reinforcing steel, can be designed for relatively low strengths. These walls not only resist the quick acting local loads but also act to help resist the deformation of the supporting frames. Furthermore the wall mass, as well as its strength, is beneficial to the frames, the walls are capable of carrying vertical floor loads in case of failure of the columns, and they may be used to replace the exterior columns. As shear members, the walls can resist large loads acting in a direction parallel to the walls.

B. Recommendations for Roof and Floor Framing

Many of the same factors, such as high-mass and lower-strength requirements, favor the use of heavy monolithic concrete for carrying the vertical load on the roof slabs. The roof also acts, in the plane of the slab, as a beam or shear member in distributing the wall loads to the frames and side-walls. In this action the heavier types of monolithic construction offer high buckling resistance to combined loads in the plane of the slab and bending transverse to the slabs. The heavy slabs also resist radiation and can offer protection against fragments of high explosive.

Light floor and roof systems, such as thin concrete slabs on bar joist or tin pan joist construction, do not appear as satisfactory as the heavier concrete framing in strength, mass, toughness, or buckling stability. A further disadvantage of such framing is the architectural necessity for suspended ceilings which crack and fail under blast loads.

In general, assuming equal or near equal construction costs, the heavier one way and two way slabs used in concrete framing are preferable to the lighter subdivided beam and girder framing.

[redacted]

C. Recommendations for Main Framing

Reinforced concrete and structural steel are suitable for use in framing blast resistant buildings, wood offers some structural possibilities if the connections are adequate and the cross partitions are designed as structural members, but the use of brick walls, even when reinforced, does not appear advisable for use in major structures.

The steel and concrete types each have certain advantages and disadvantages with respect to each other. The steel framing is capable of developing higher resisting moments per unit of displaced volume and the connections can possibly be stronger and more rigid, not having the difficulties with bond that is experienced in the design of concrete frame structures.

Steel framing seems generally more expensive than reinforced concrete frames and if fireproofing is required the cost of steel frames will probably be appreciably higher in almost all cases.

The use of concrete partitions and cross walls as shear walls to replace rigid frames of either steel or concrete may prove the most economical of all framing types designed to resist blast pressures. While the action of these members is quicker and the load factors are higher than in the frames,the advantage and economy of carrying loads by direct stress rather than by flexure is apparent.

These recommendations on desirable framing are based on using the severe requirements of this particular test structure as the criteria for the comparison of different materials, of members, and framing types. While generally true for any intensity of loading, it might not be possible to make some of the above recommendations in as positive a manner for lower values of blast intensity where the relative costs of the different competitive materials may be less distinct. The recommended framing methods and materials thus hinge to some extent, on the maximum pressure for which resistance is to be provided.

An example of this might be as follows. The columns and beams of conventional buildings are formed of substantial members which will provide considerable resistance against lateral loads if the details are made adequate to develop their strength. This framing can be made adequate for light blast loads with a minimum of expense and attention. The addition of structural shear walls might be unnecessarily conservative and expensive in this case.

Without going into details which are beyond the scope of this contract, it might be expected that certain large geographical areas are not likely to be concerned with blast pressures of any kind,as these particular areas may not be of sufficient strategic importance to justify such attacks. In other areas dispersion of facilities may discourage attack or at least limit the damage to a small group of buildings. The individual units in this case may require protection in proportion to the probability of their exposure and individual importance. It is conceivable, however, that a certain number of other areas may contain buildings located near enough to a probable target center so that some degree of blast pressure may be expected. The target center, in this case may immediately relate all other buildings

[redacted]

in the vicinity to a certain expected scale of intensity of blast pressure depending directly on the distance of each such building from the expected zero point and on the probable limits of accuracy in dropping the bomb. Considering the variation in the degree of blast resistance required by the various buildings, several criteria of design will be needed and these will have a material effect on any specific proposals and recommendations for framing. To this end an approach toward the establishment of general criteria should be made on the basis of a careful study of optimum design.

[redacted]

APPENDIX I

SUMMARY OF PRESSURE CURVES

CONTENTS

[redacted]

APPENDIX I

CONTENTS

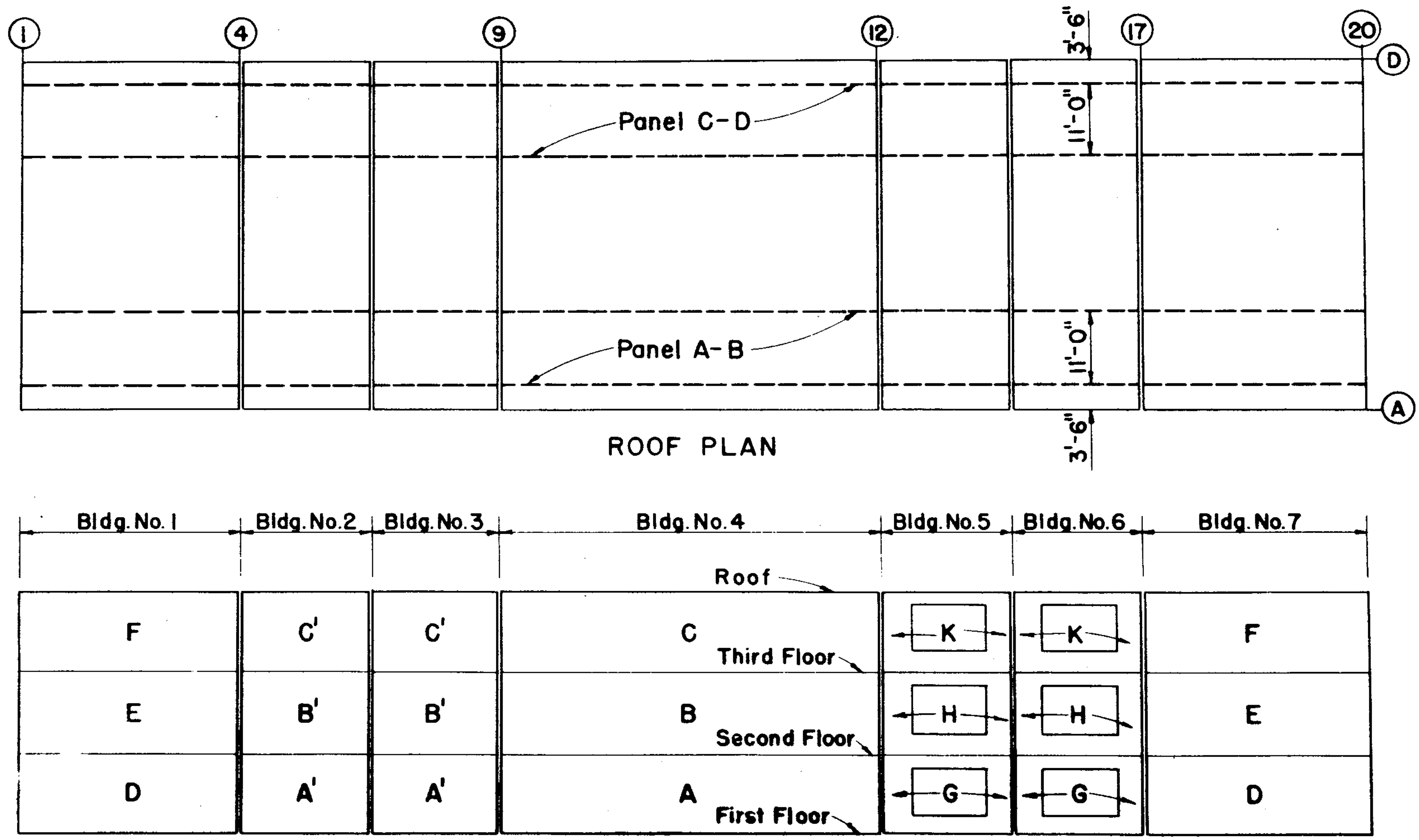
(Cont'd)

LEGEND

Figures 1 to 23 and 25 to 48

First and Second Issue
Third Issue
Fourth Issue
Design Group - Fifth Issue
Final Group - Sixth Issue

[redacted]

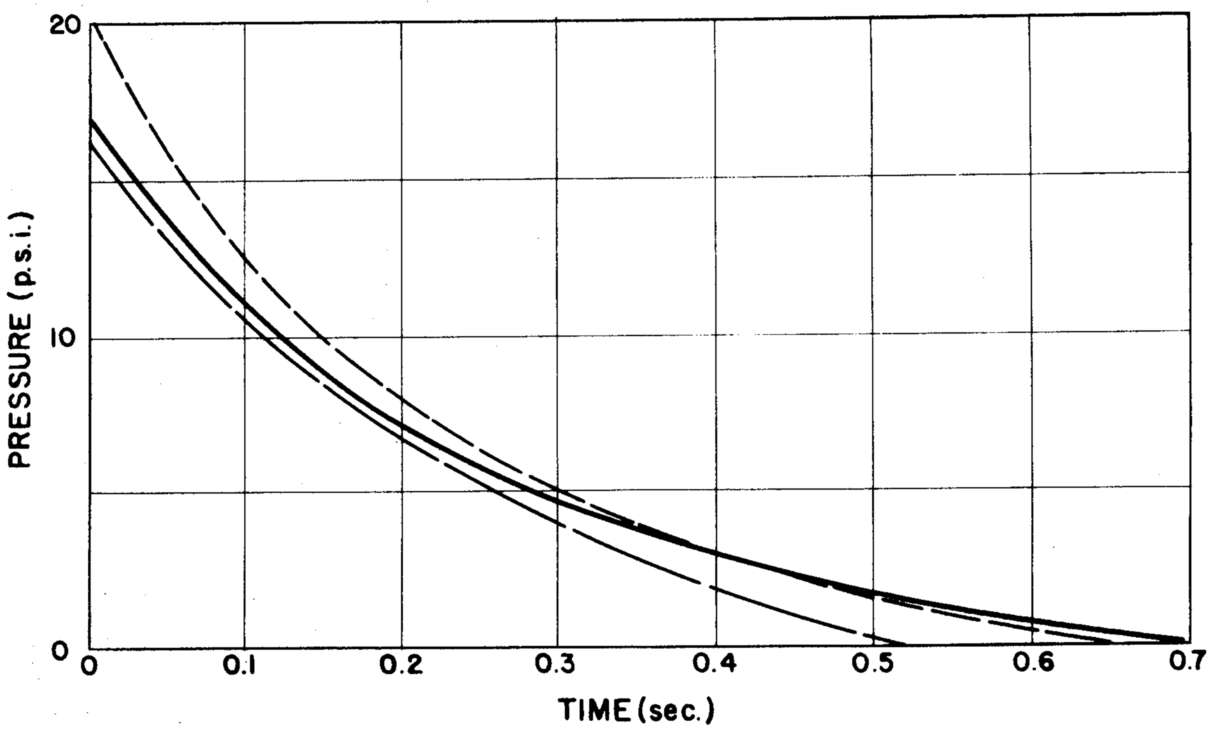


FRONT ELEVATION-(REAR ELEVATION OPPOSITE HAND)

LOCATION DIAGRAM FOR PRESSURE CURVES

FIG. AI-1

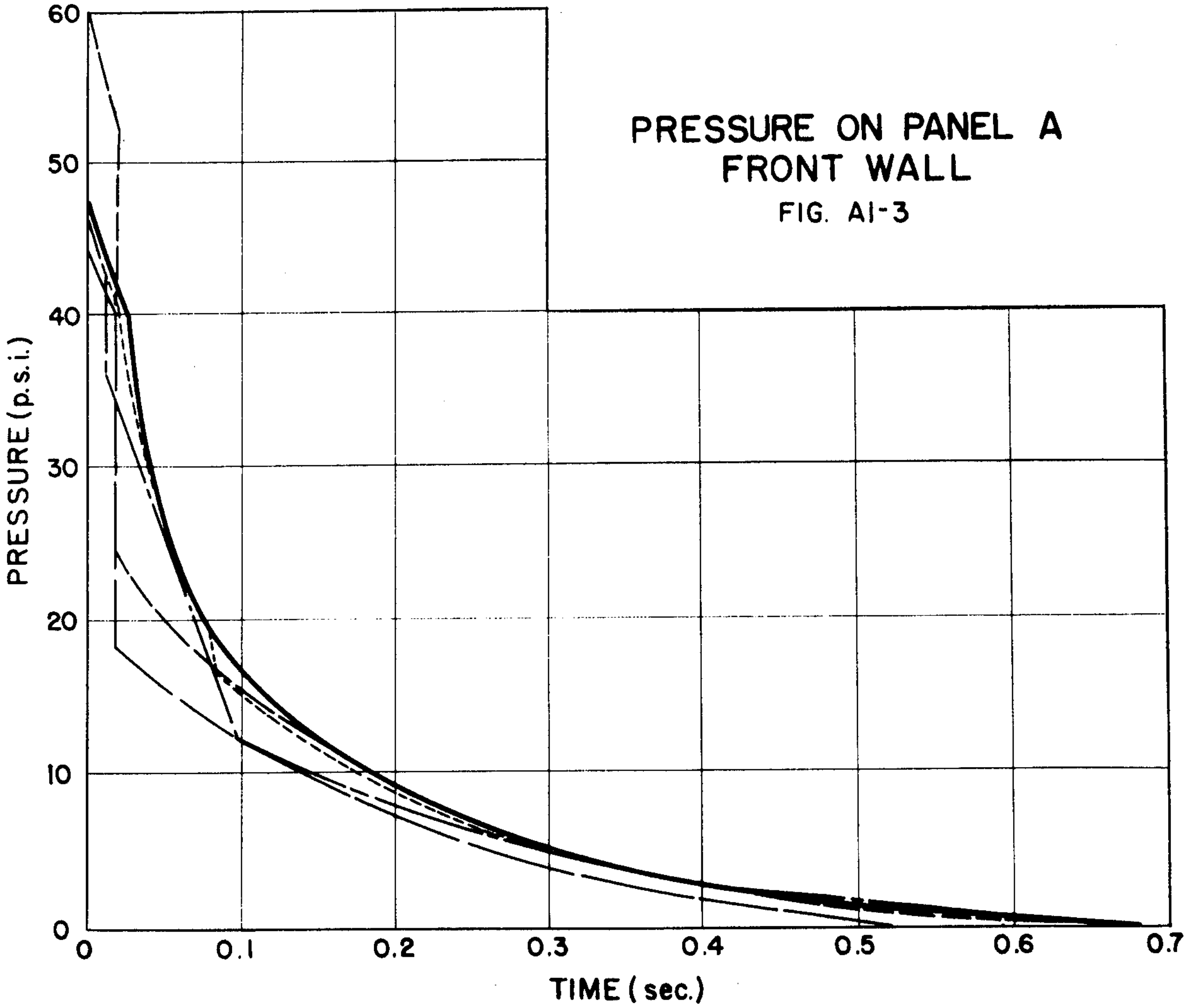
[redacted]



PRESSURE ON SIDE WALLS
(MODIFIED FRIEDRICH'S EQUATION)

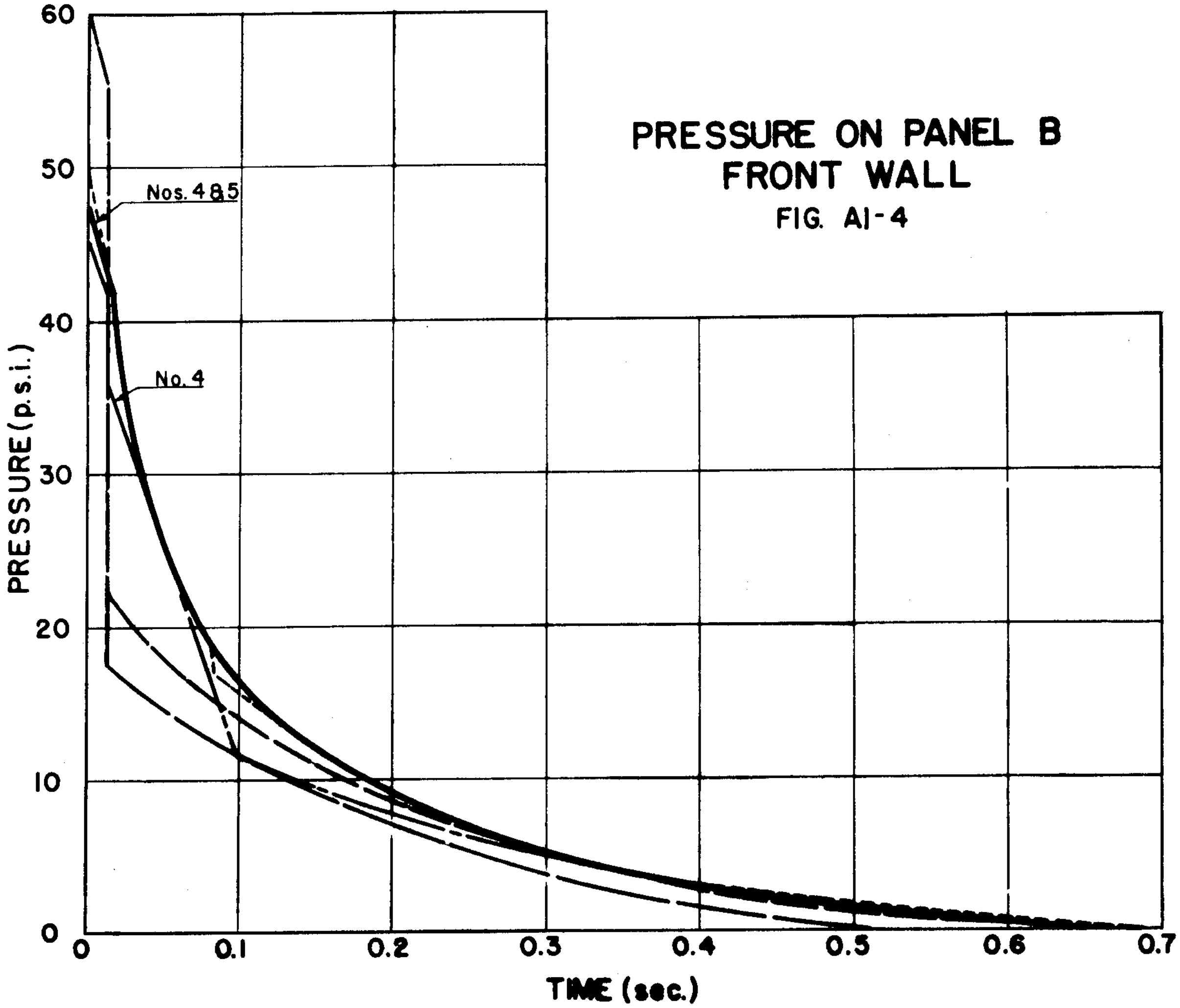
FIG. AI-2

[redacted]



PRESSURE ON PANEL A
FRONT WALL
FIG. A1-3

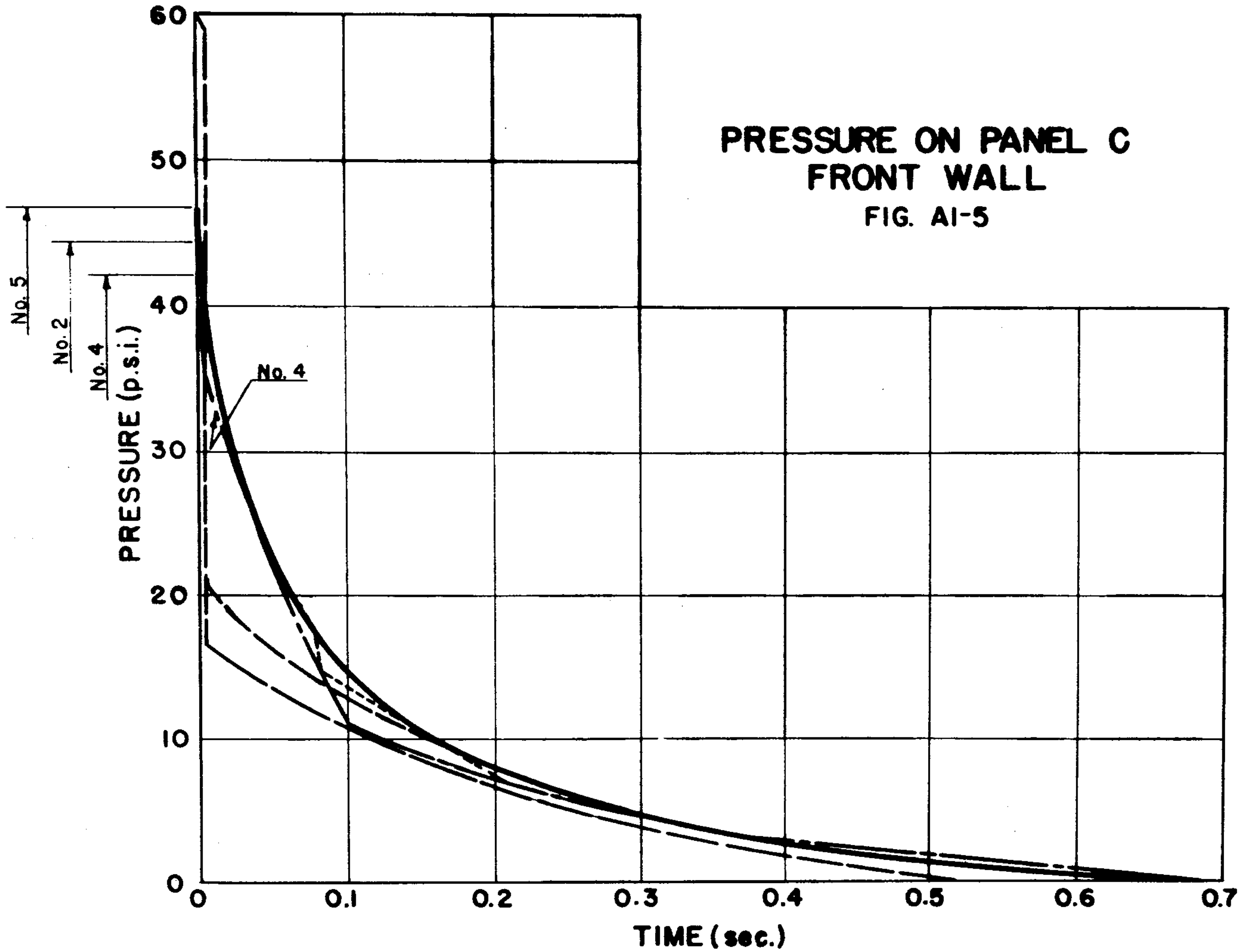
[redacted]

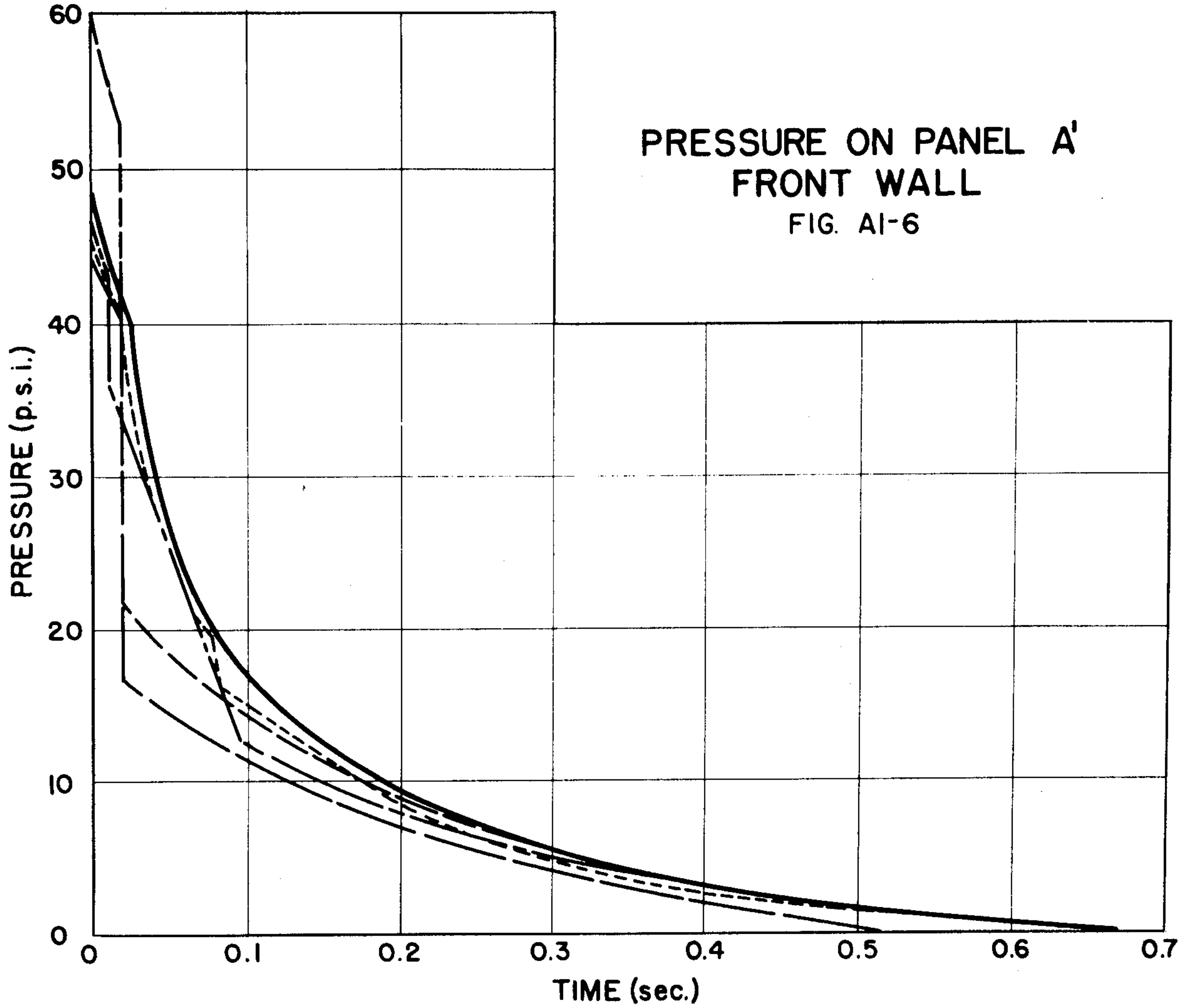


PRESSURE ON PANEL B
FRONT WALL
FIG. A1-4

[redacted]

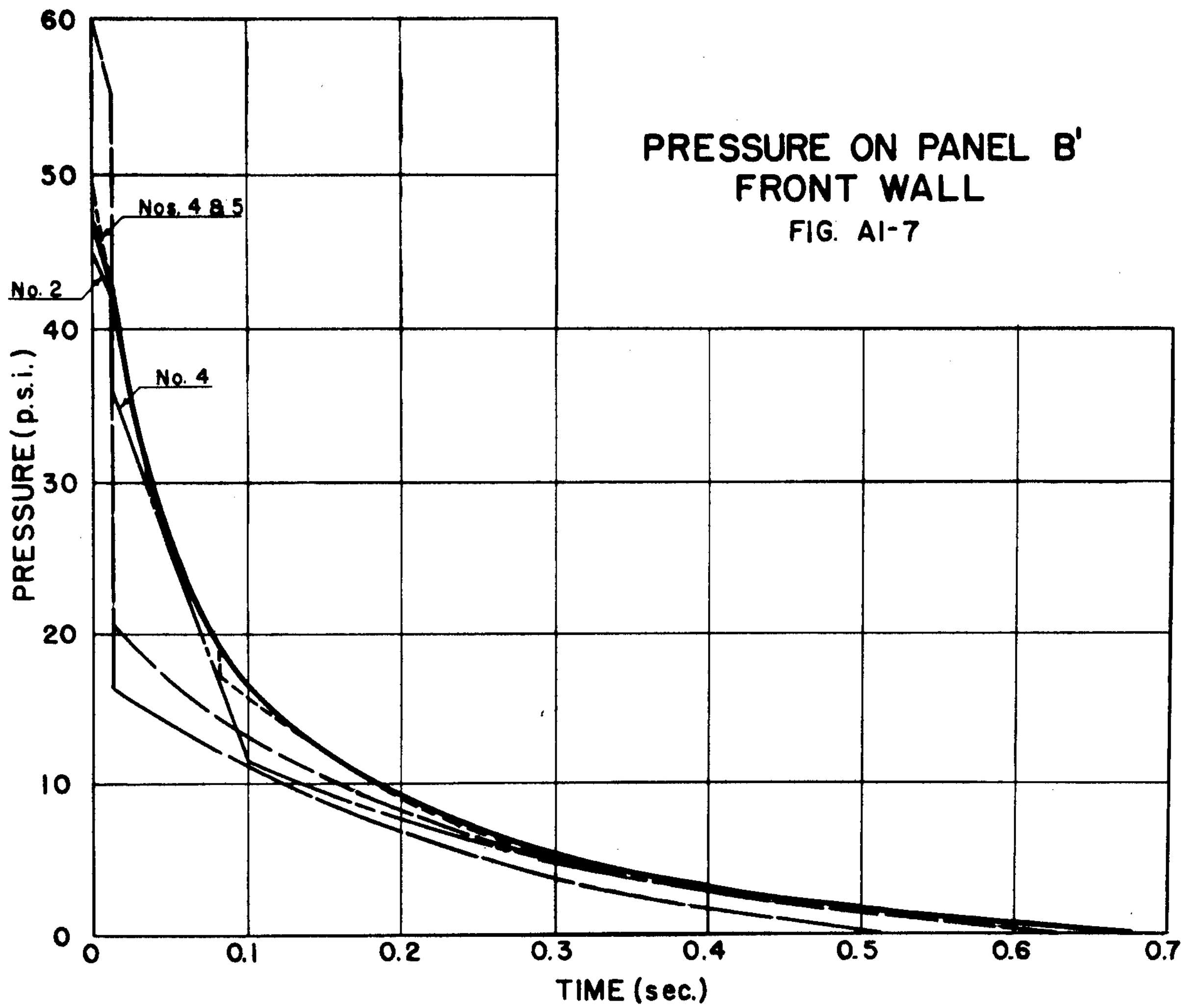
PRESSURE ON PANEL C
FRONT WALL
FIG. AI-5





PRESSURE ON PANEL A'
FRONT WALL
FIG. A1-6

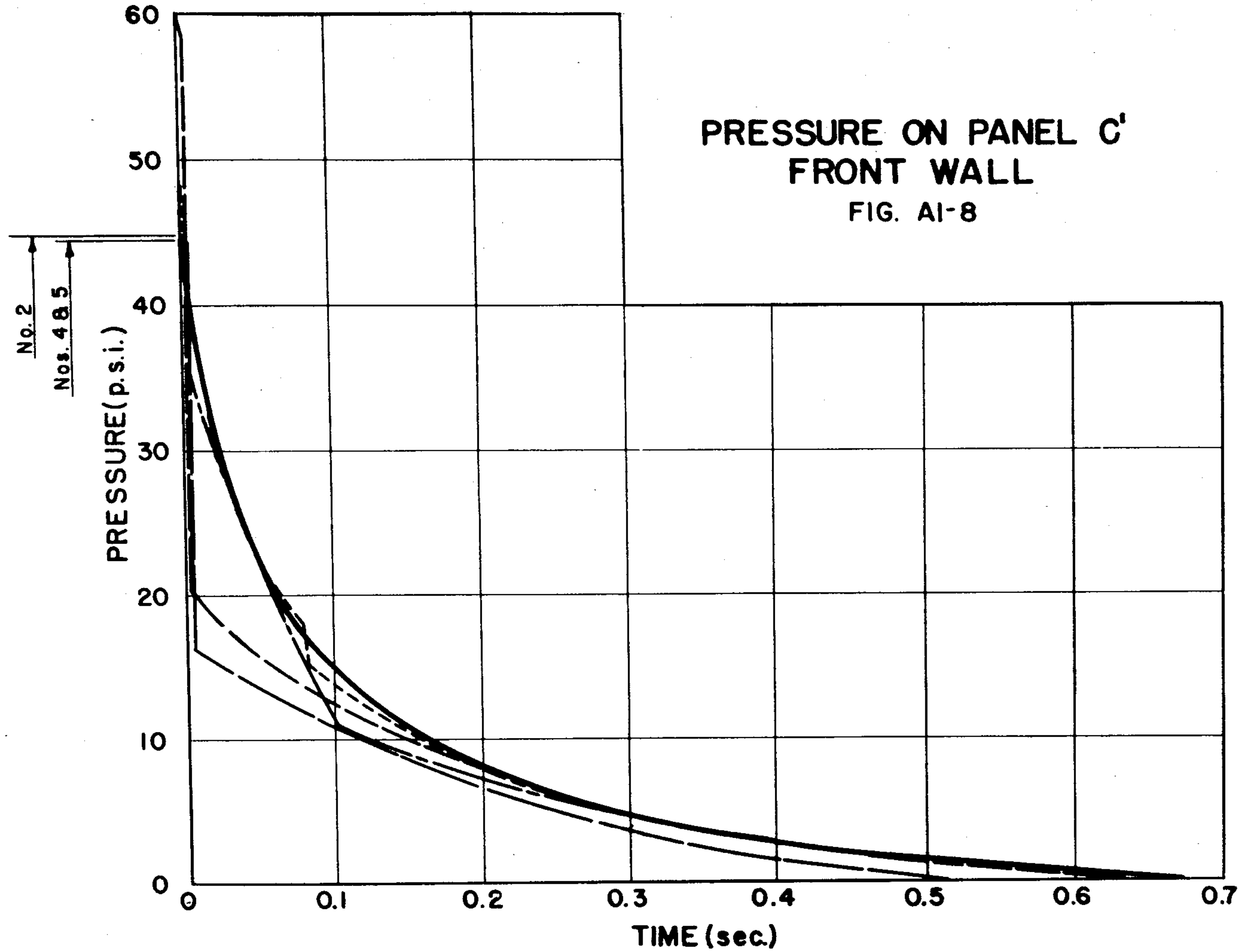
[redacted]



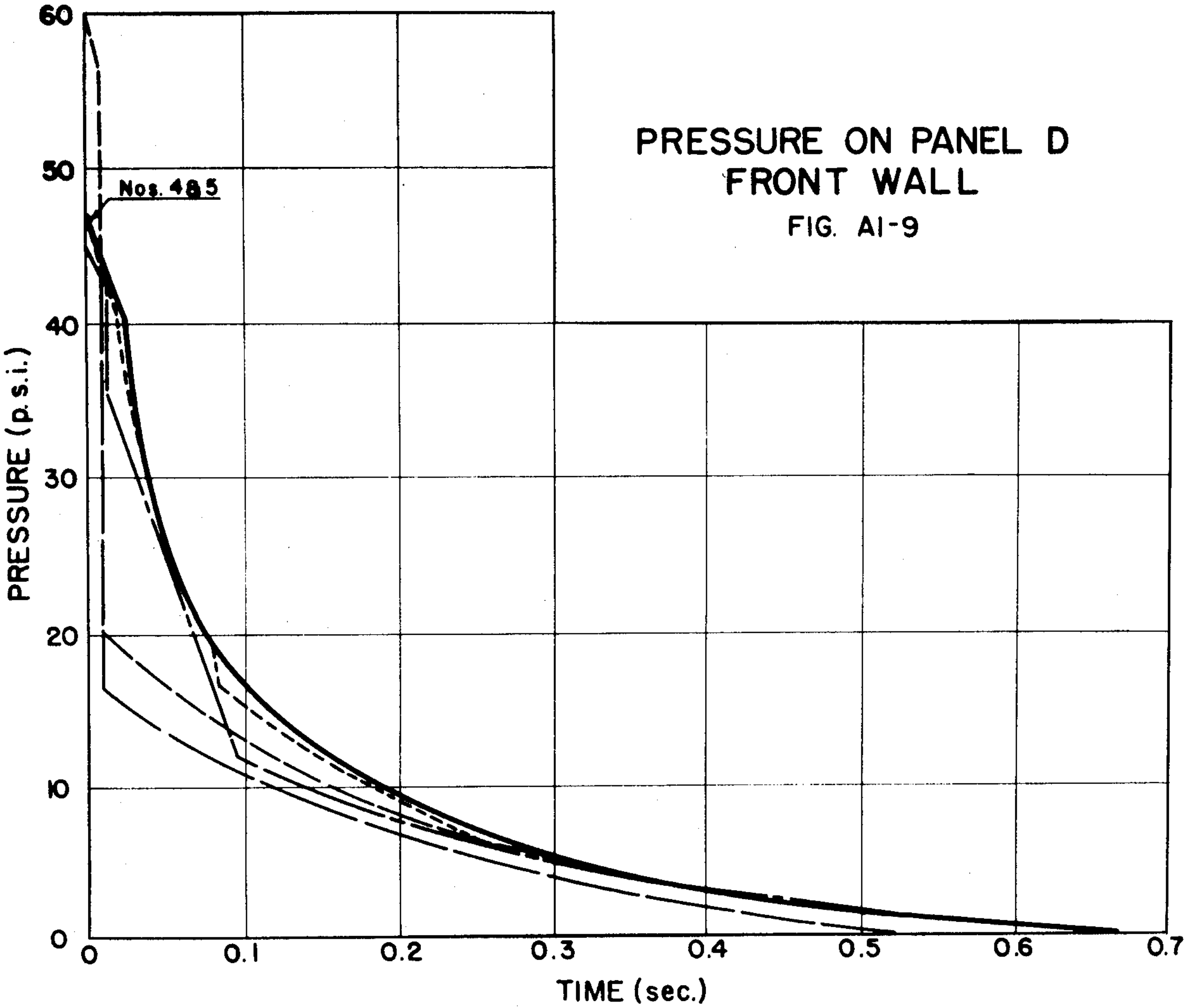
PRESSURE ON PANEL B'
FRONT WALL
FIG. AI-7

[redacted]

PRESSURE ON PANEL C'
FRONT WALL
FIG. AI-8

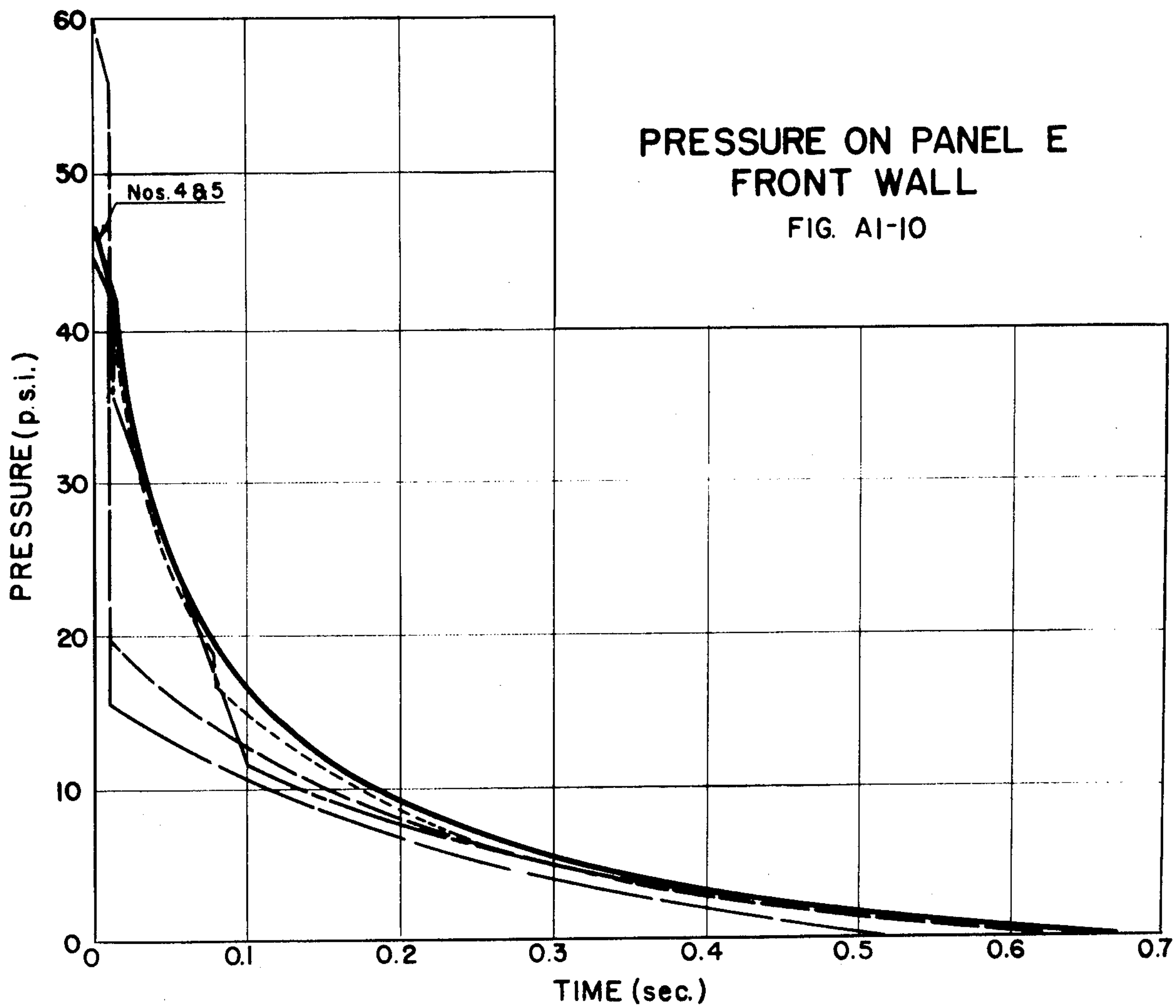


[redacted]

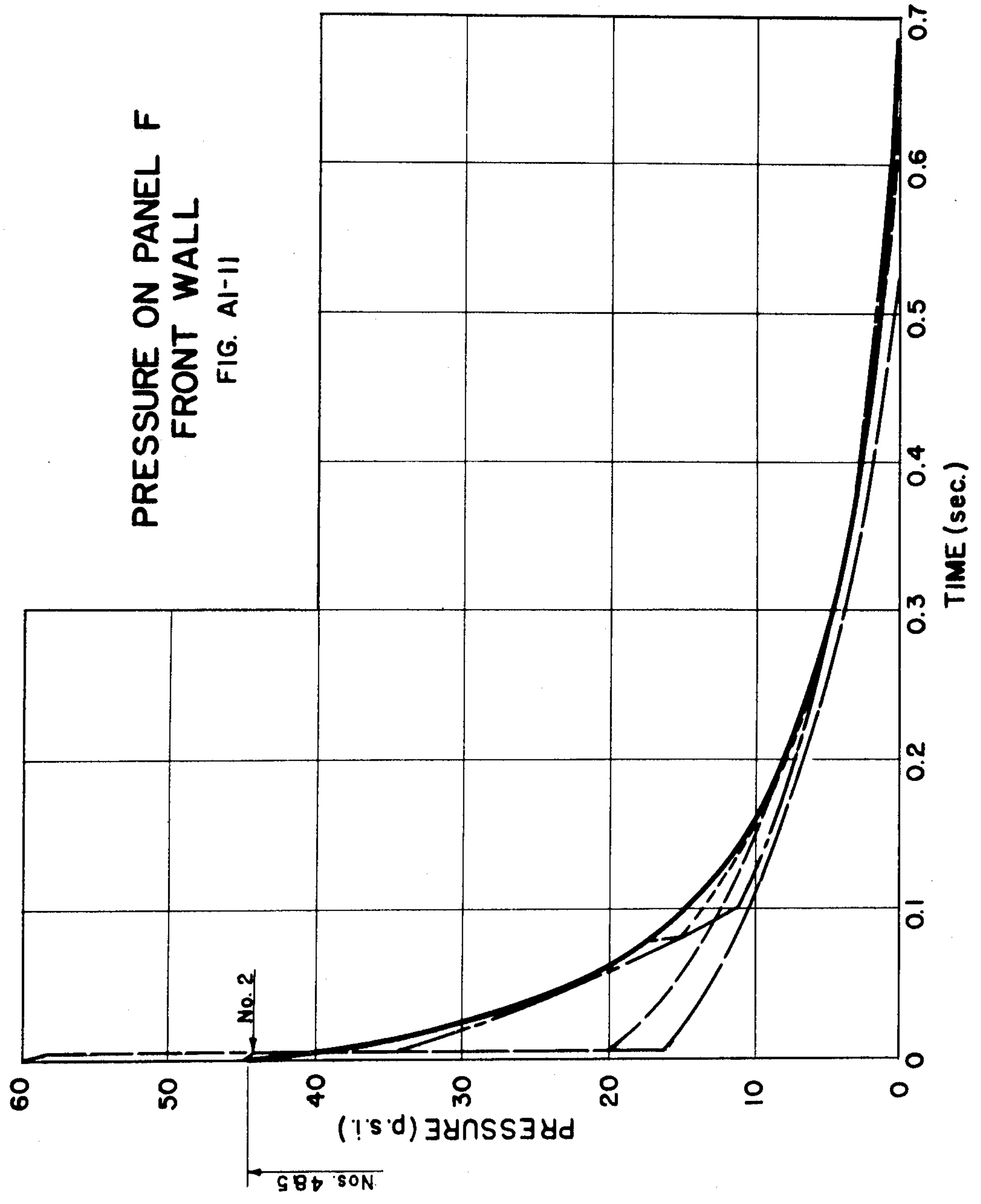


PRESSURE ON PANEL D
FRONT WALL
FIG. A1-9

[redacted]

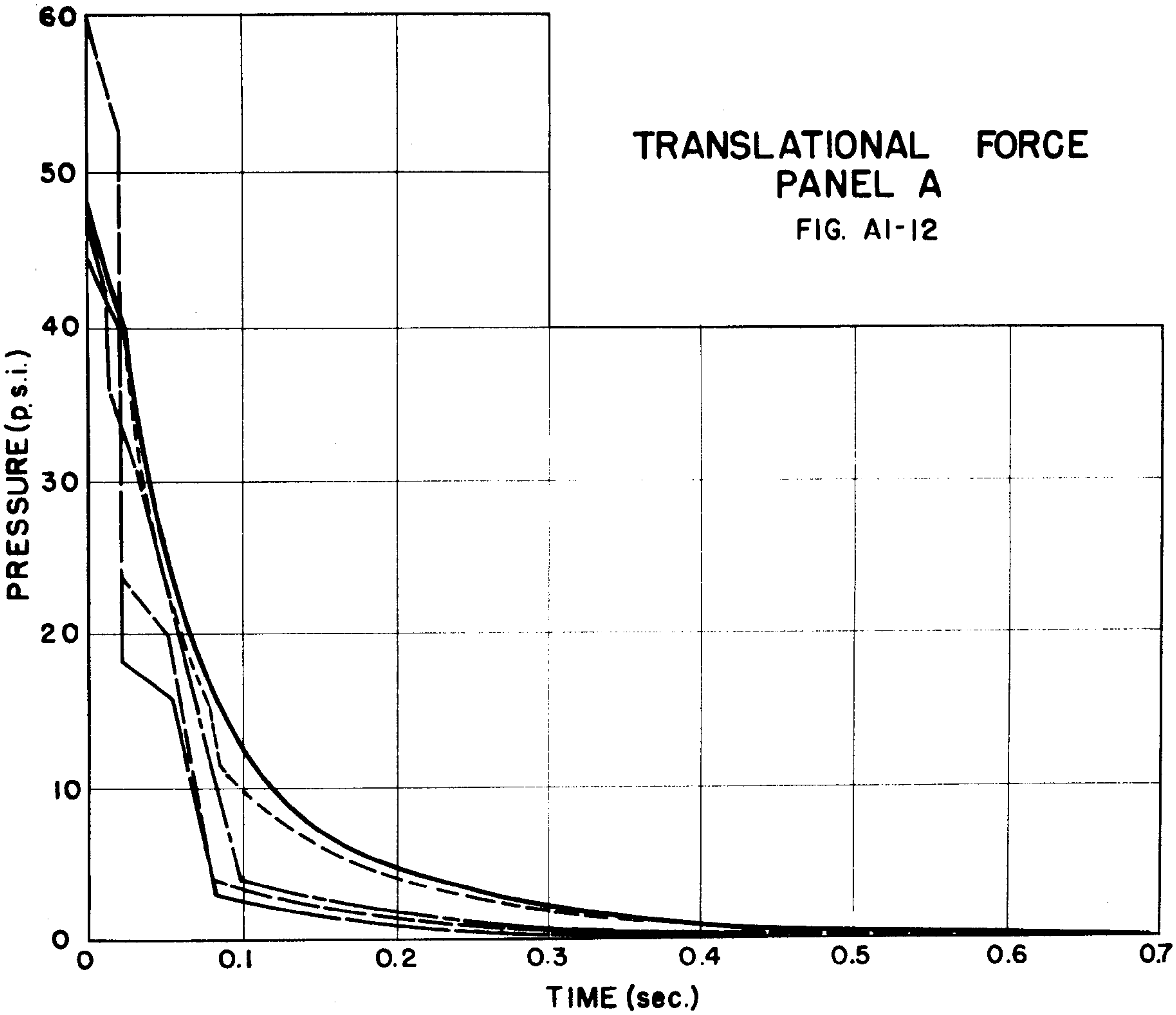

PRESSURE ON PANEL E
FRONT WALL
FIG. A1-10
Nos. 4 & 5
PRESSURE (p.s.i.)
TIME (sec.)
60
50
40
30
20
10
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

[redacted]



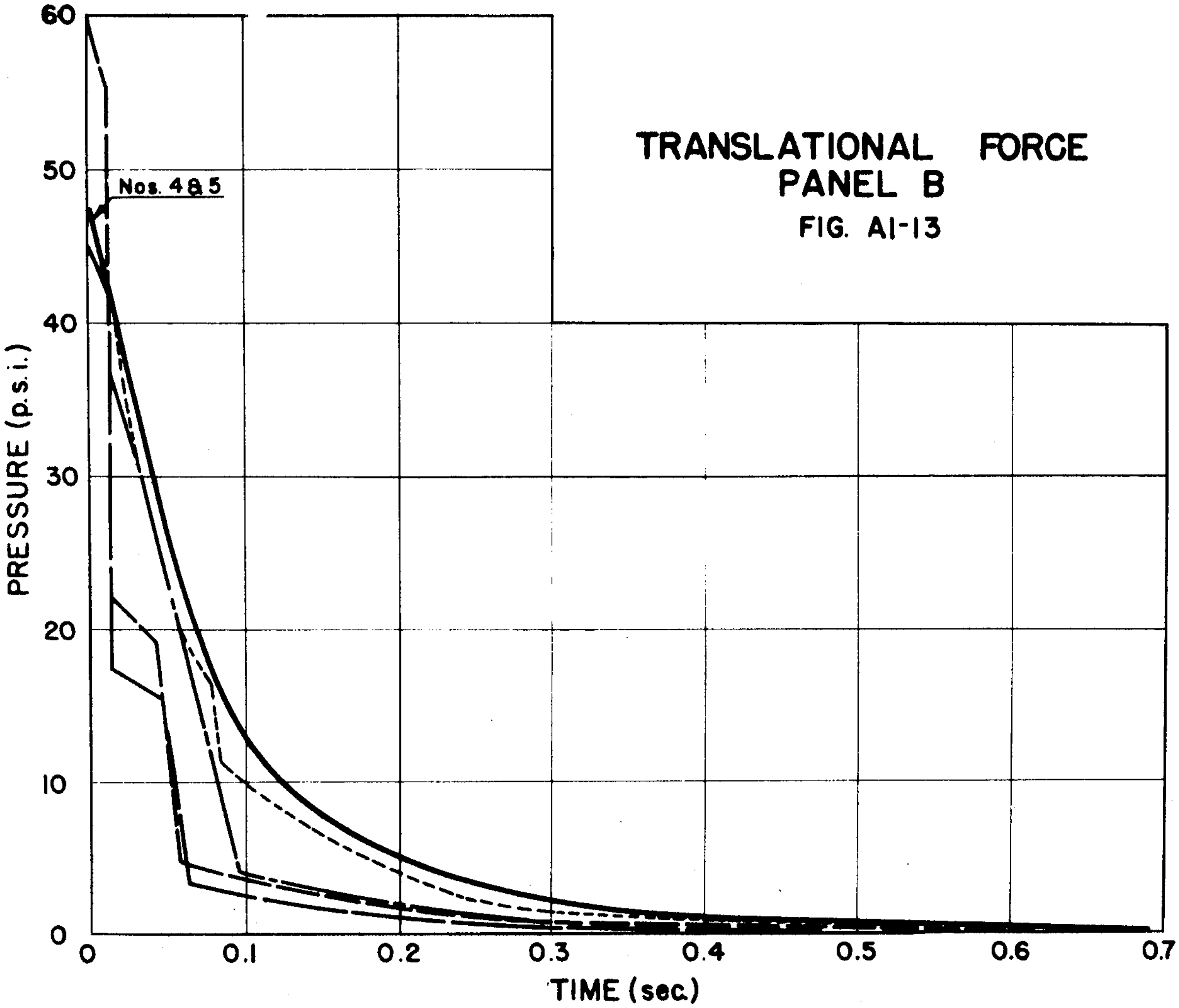
PRESSURE ON PANEL F
FRONT WALL
FIG. AI-11

[redacted]



TRANSLATIONAL FORCE
PANEL A
FIG. A1-12

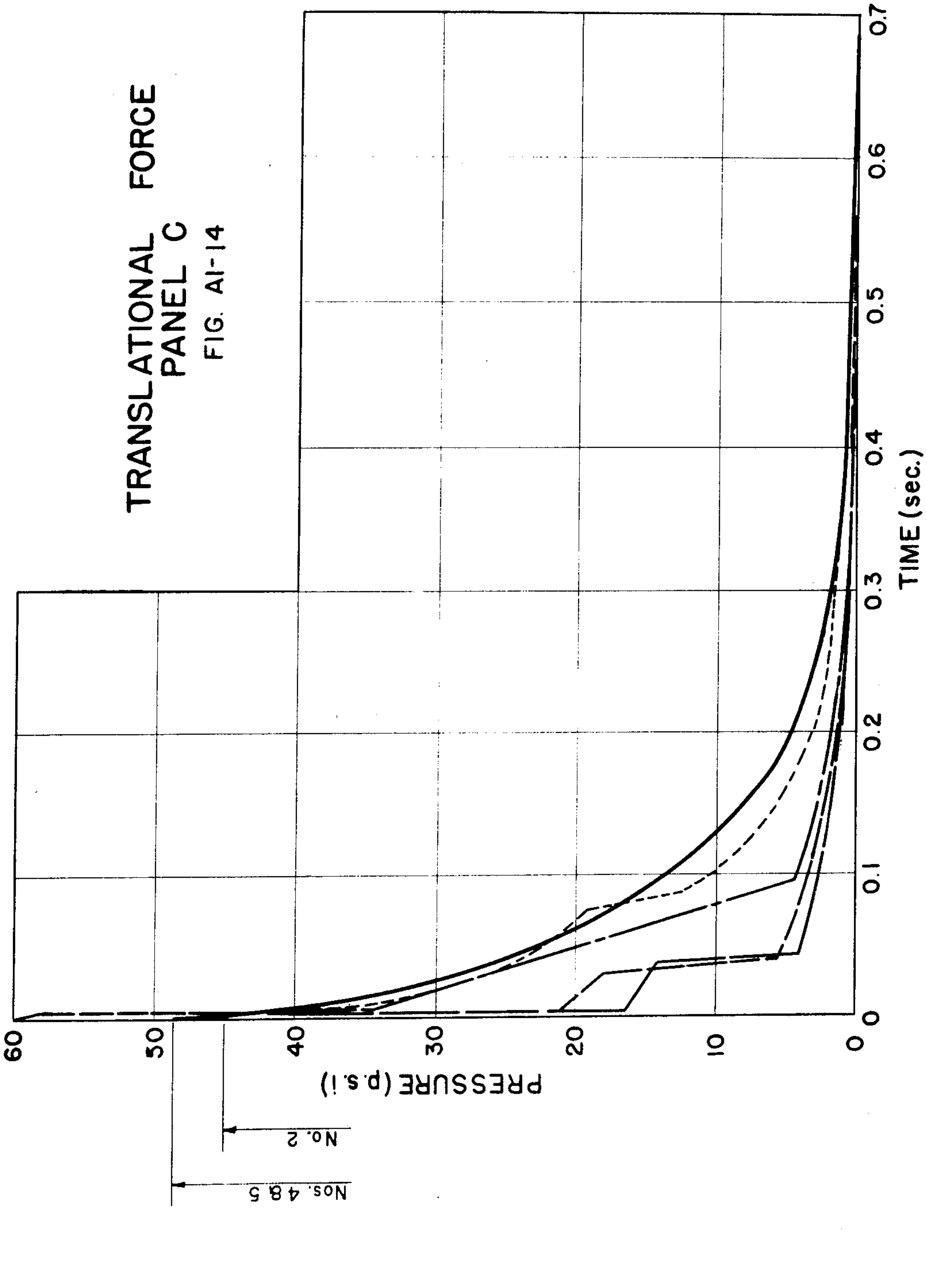
[redacted]



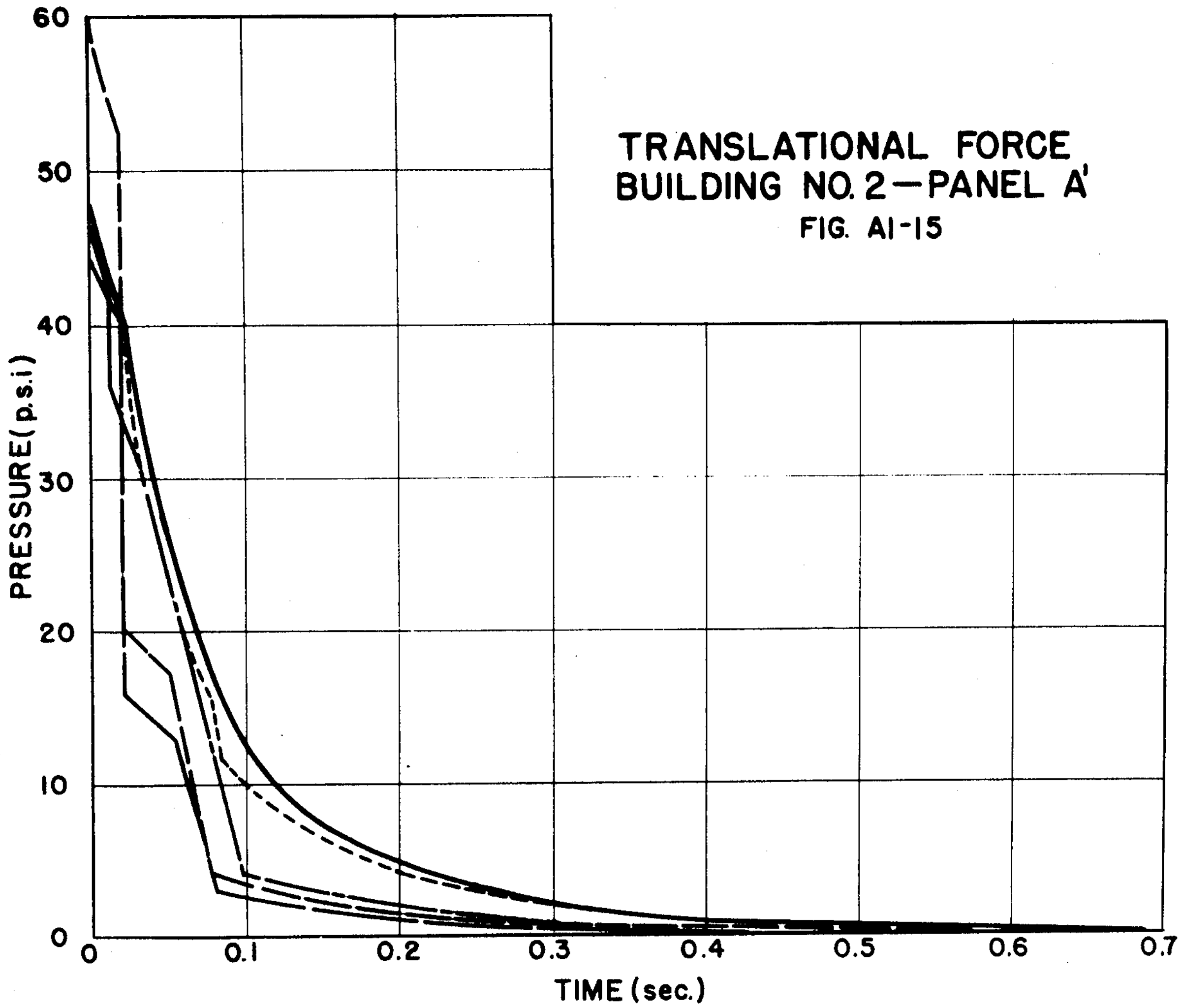
TRANSLATIONAL FORCE
PANEL B
FIG. A1-13

[redacted]

TRANSLATIONAL FORCE
PANEL C
FIG. A1-14

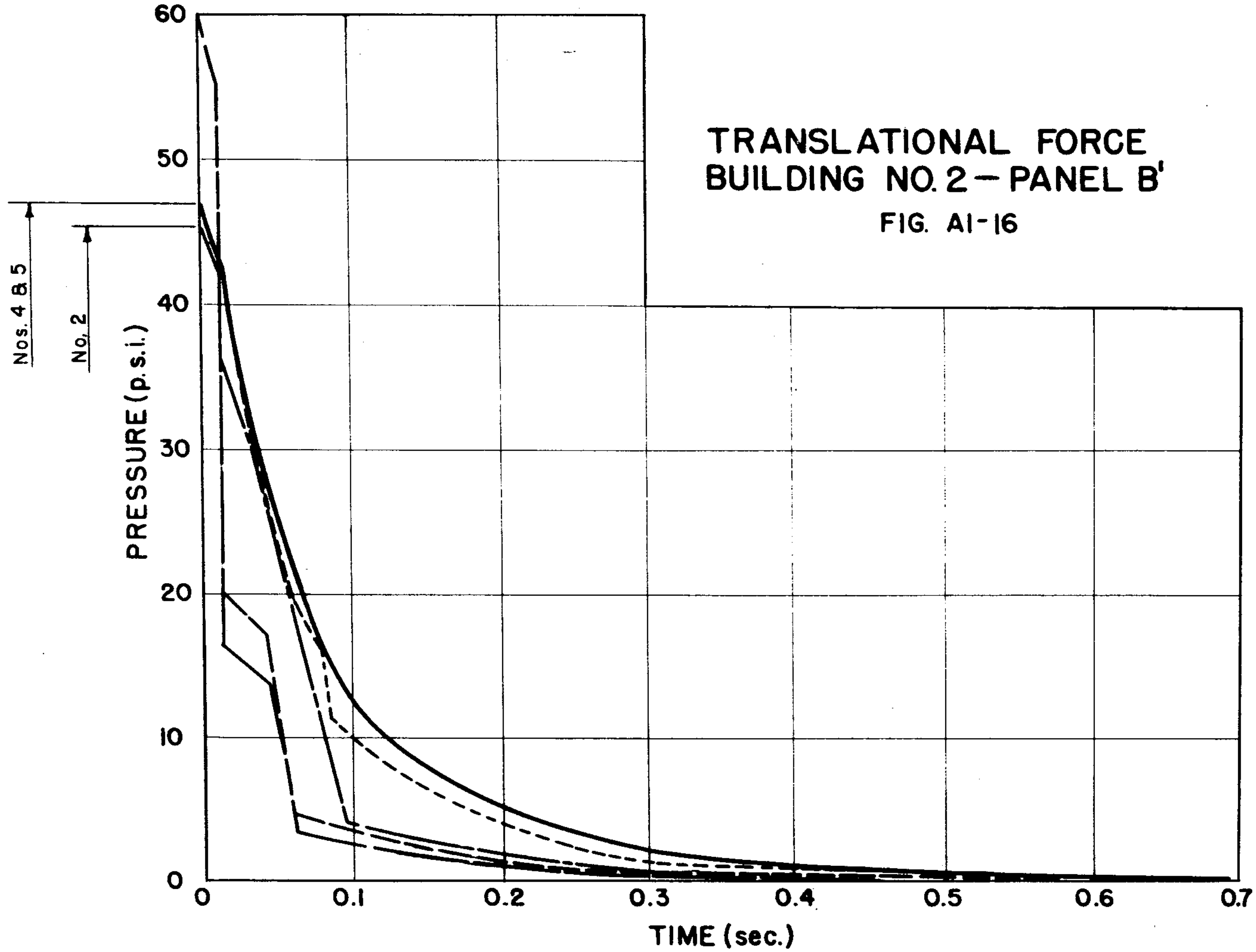


[redacted]



TRANSLATIONAL FORCE
BUILDING NO. 2—PANEL A'
FIG. A1-15

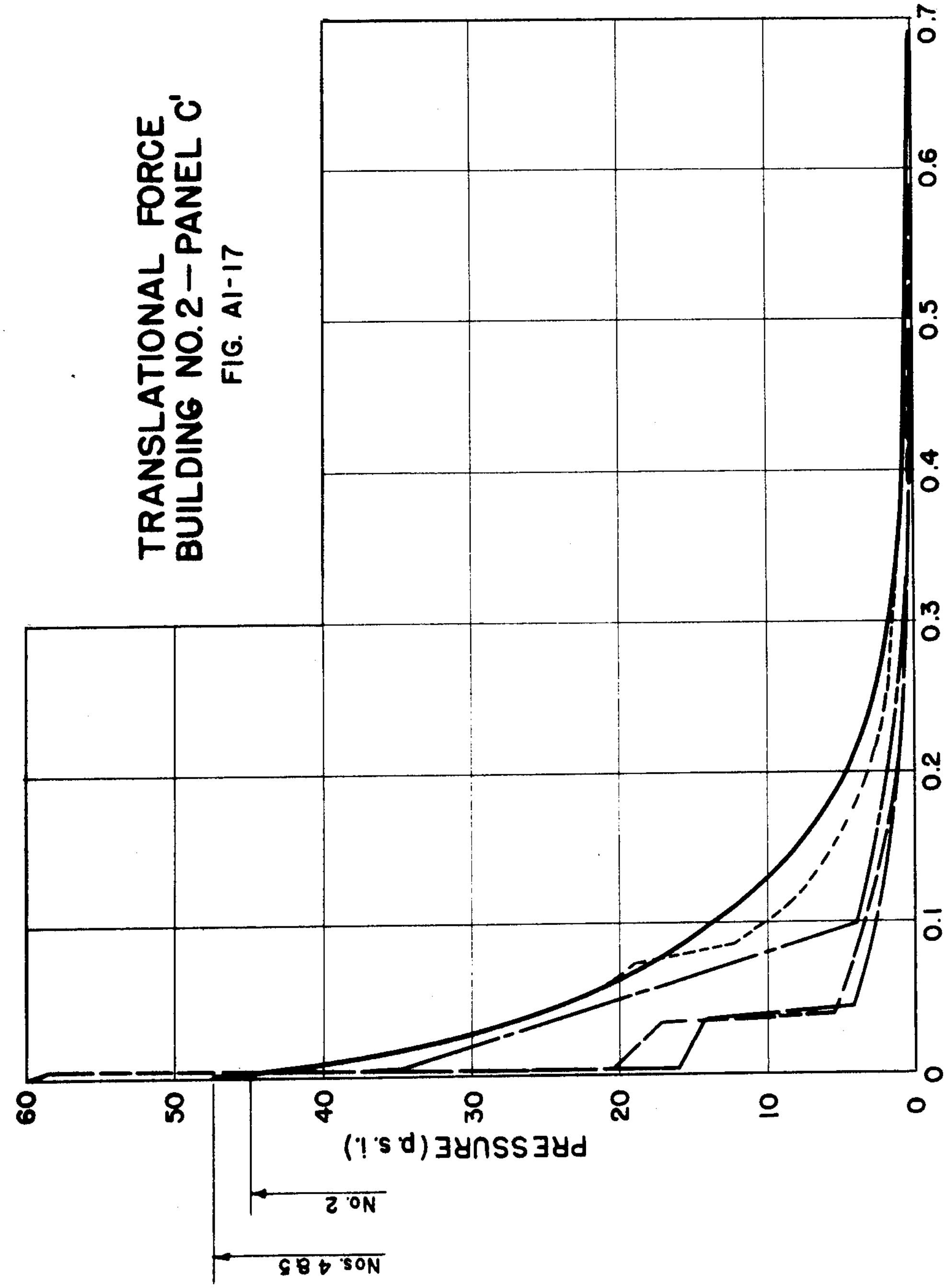
[redacted]



TRANSLATIONAL FORCE
BUILDING NO. 2 — PANEL B'

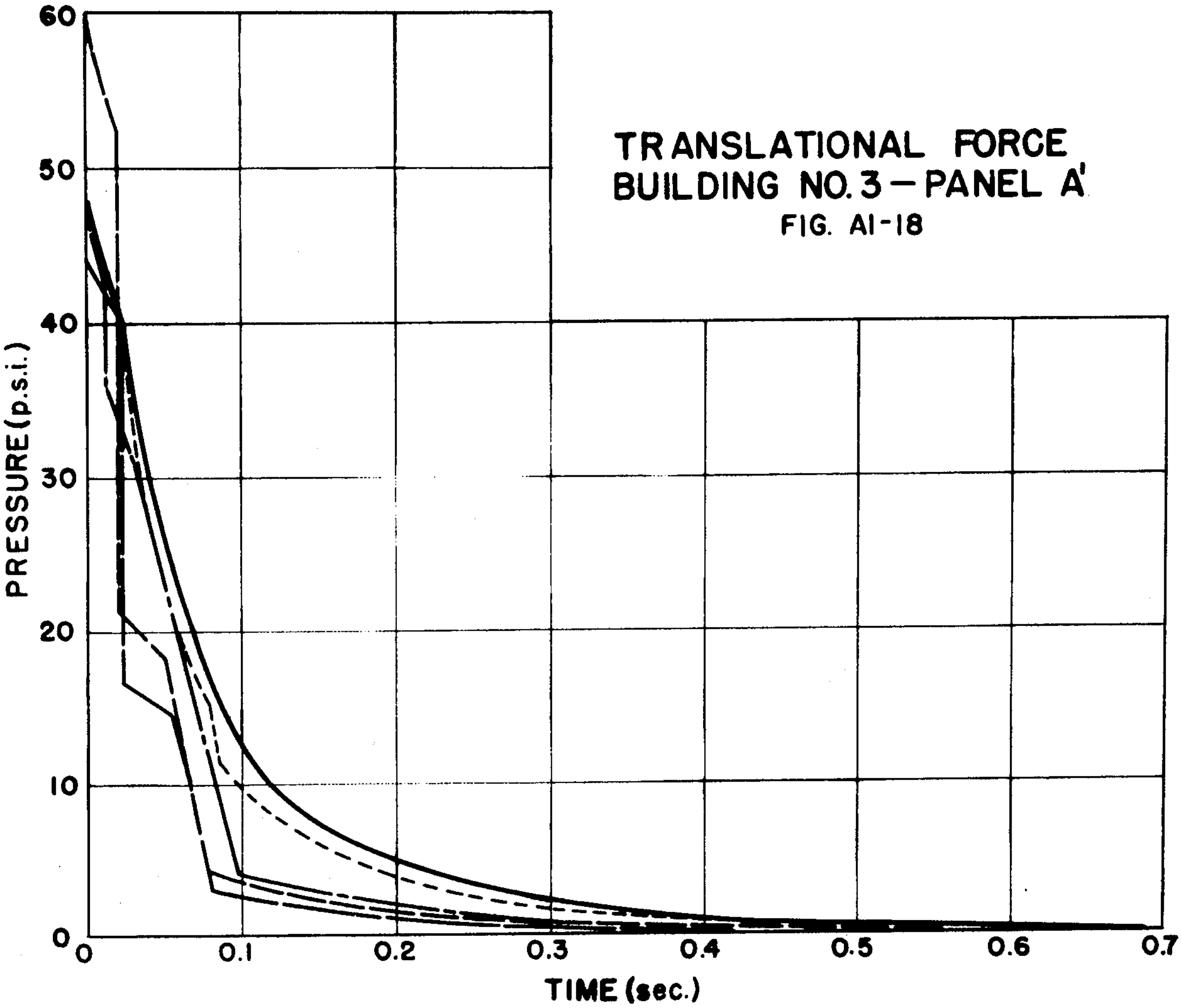
FIG. A1-16

[redacted]



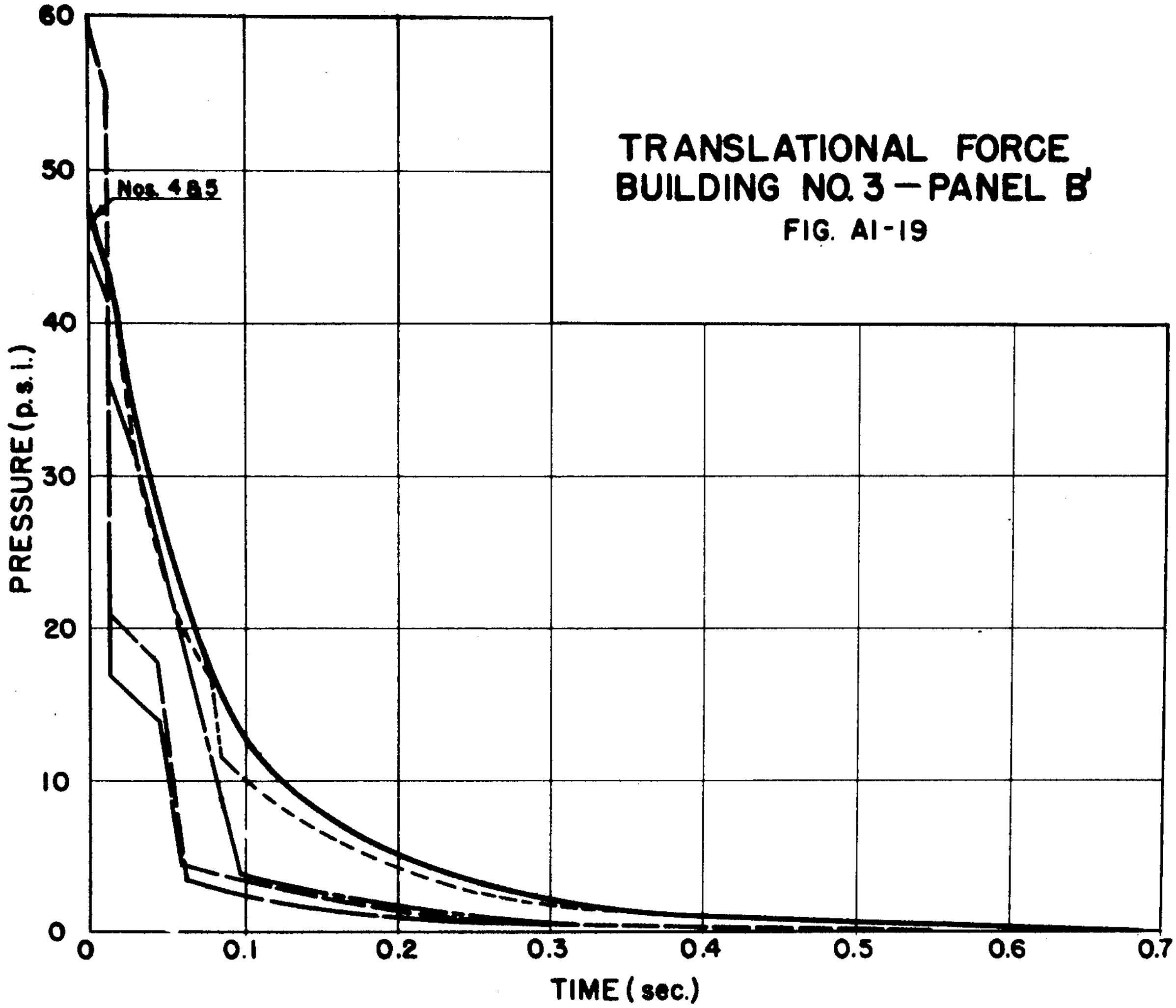
TRANSLATIONAL FORCE
BUILDING NO. 2—PANEL C'
FIG. A1-17

[redacted]

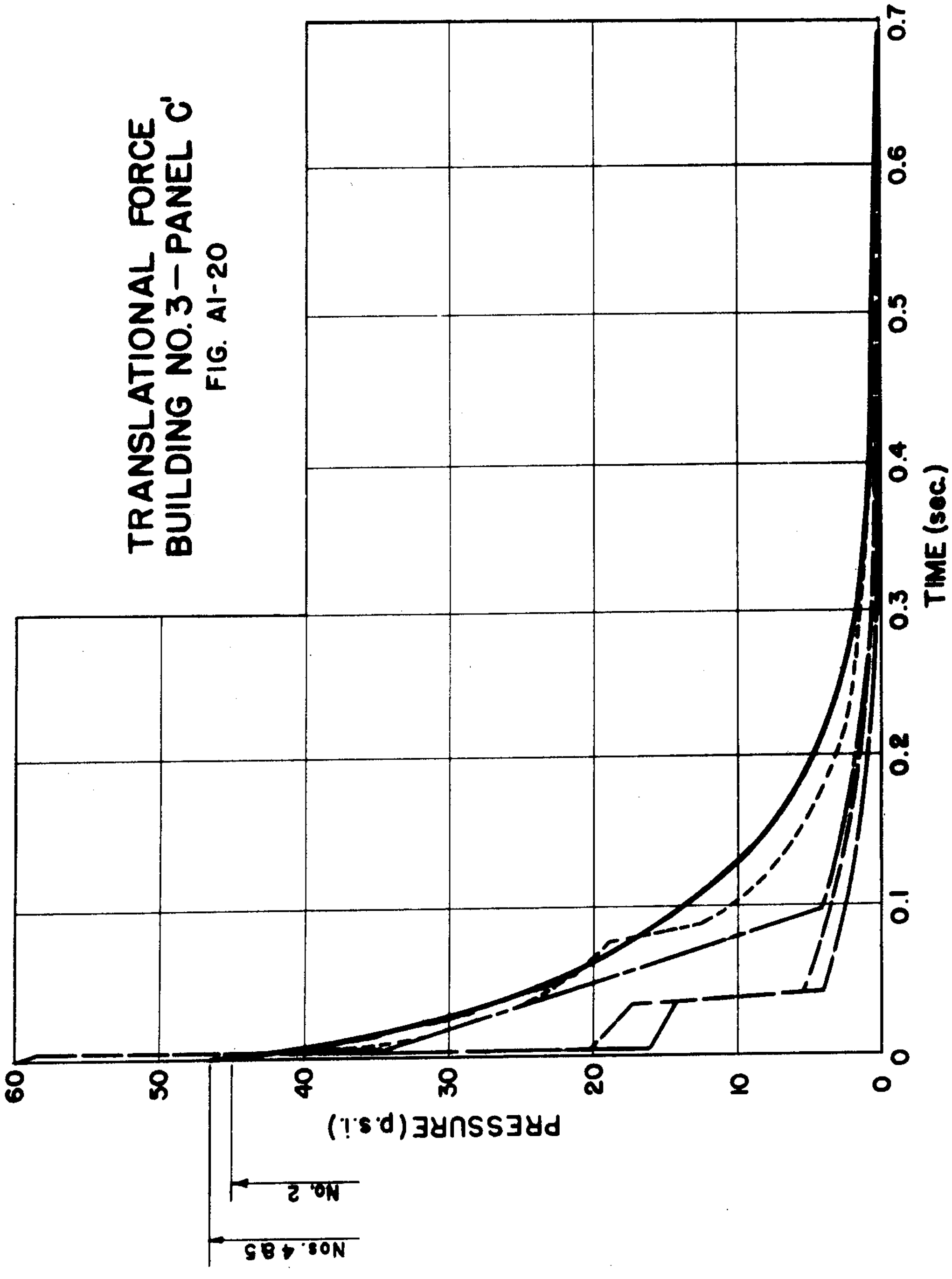


TRANSLATIONAL FORCE
BUILDING NO. 3 — PANEL A'
FIG. A1-18

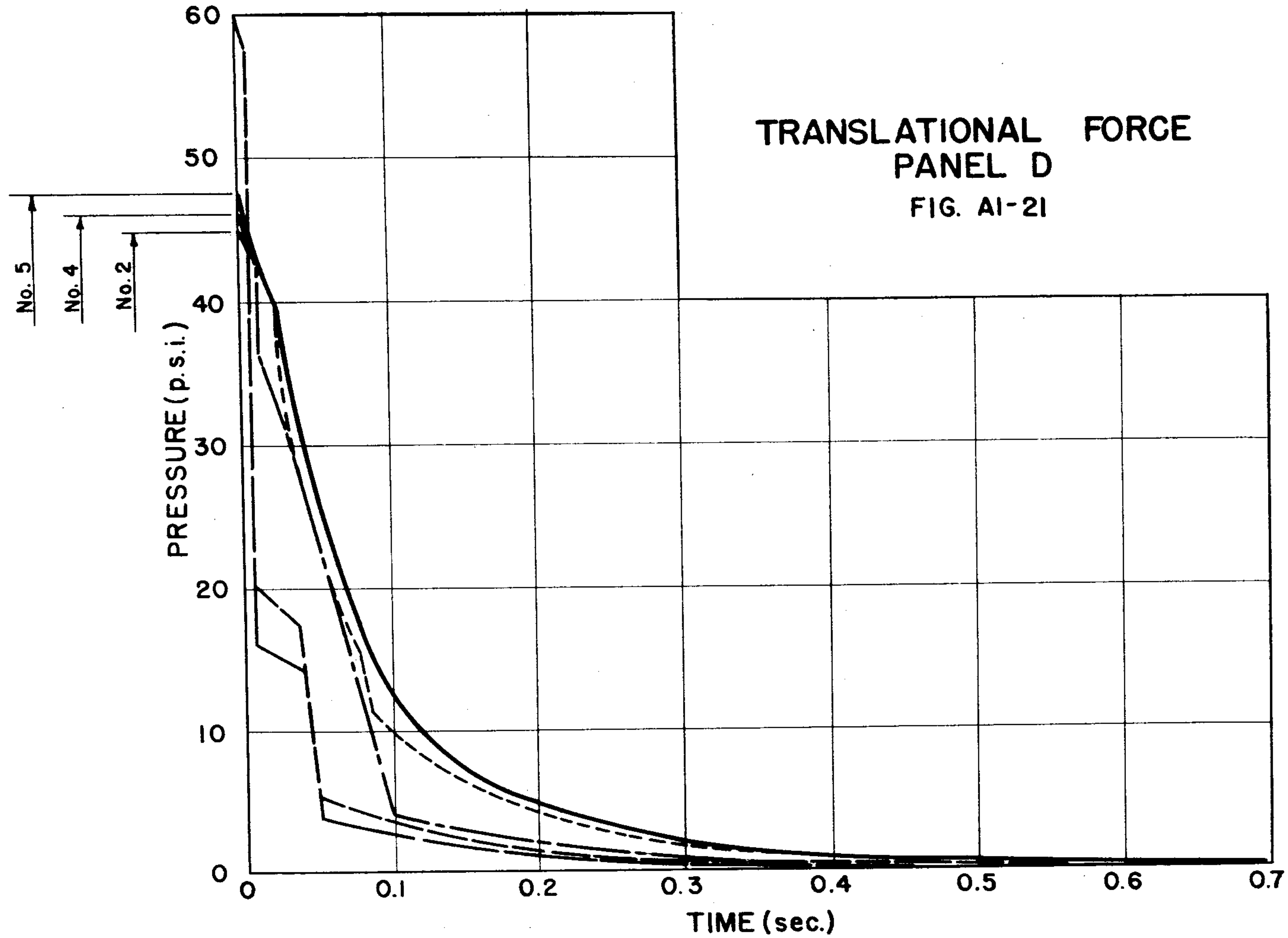
[redacted]


TRANSLATIONAL FORCE
BUILDING NO. 3 — PANEL B'
FIG. AI-19
Nos. 4 & 5
PRESSURE (p.s.i.)
0
10
20
30
40
50
60
TIME (sec.)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

[redacted]

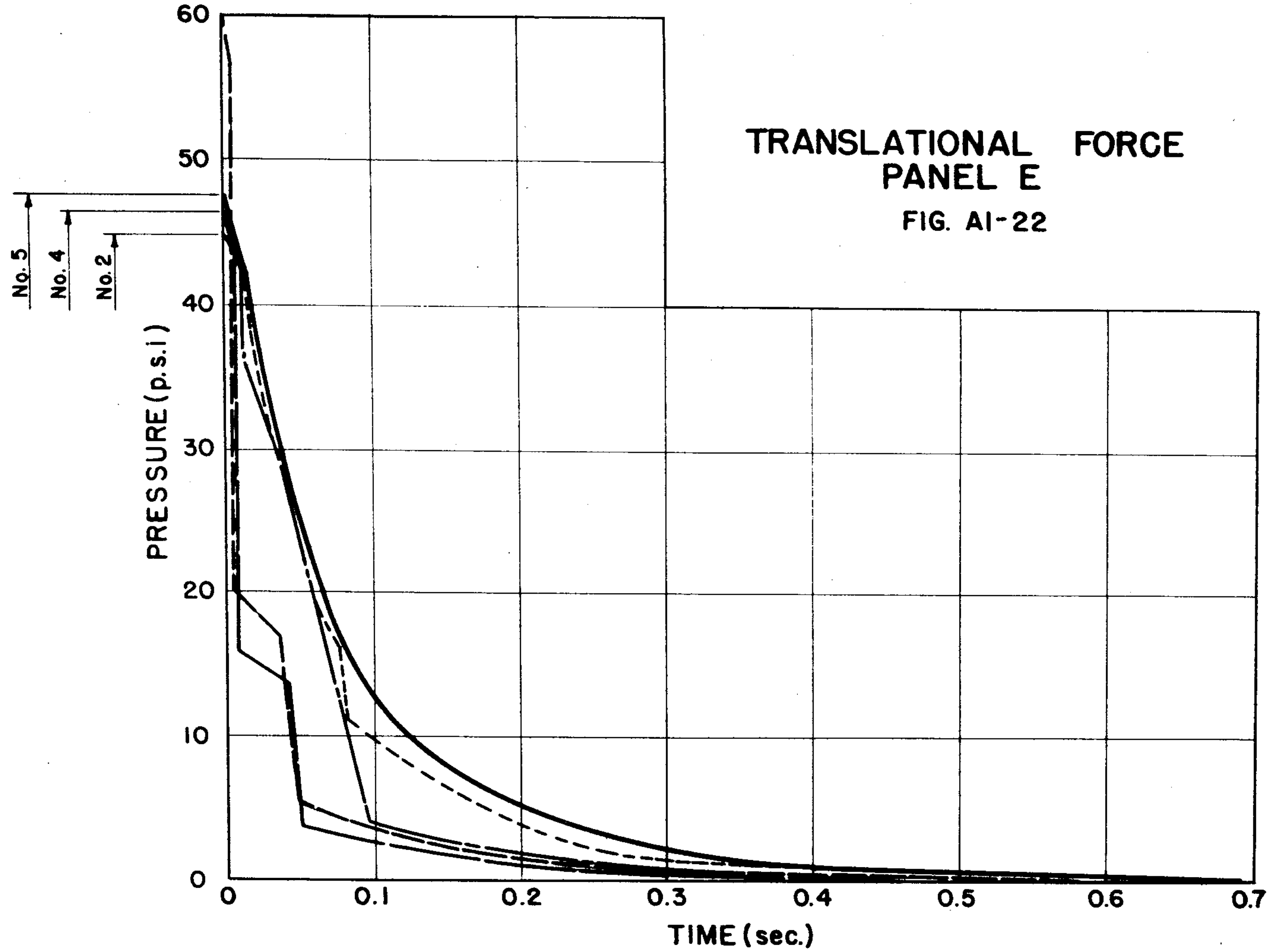

TRANSLATIONAL FORCE
BUILDING NO. 3 — PANEL C'
FIG. AI-20
PRESSURE (p.s.i.)
TIME (sec.)
No. 2
Nos. 4 & 5
0
10
20
30
40
50
60
0.1
0.2
0.3
0.4
0.5
0.6
0.7

[redacted]

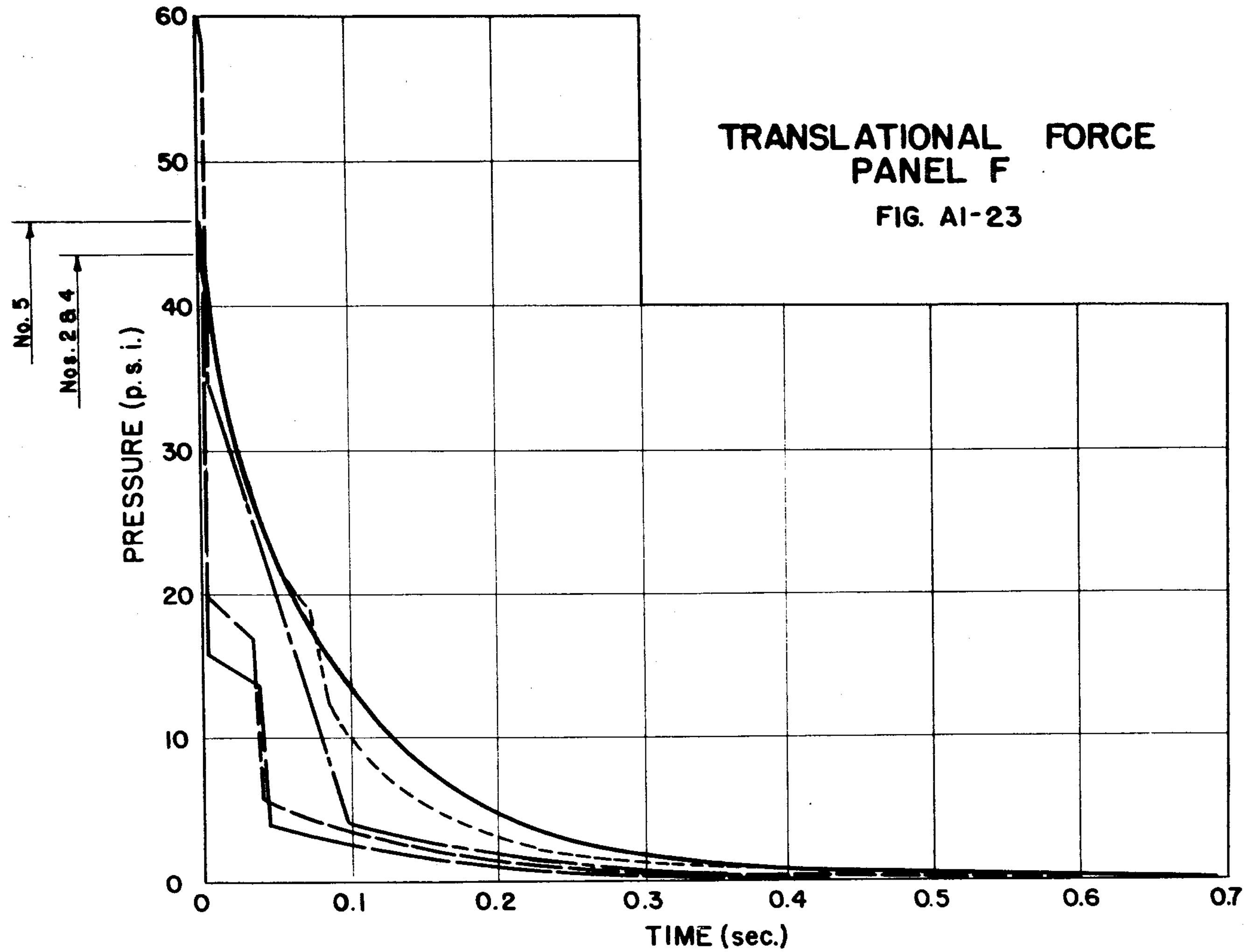


TRANSLATIONAL FORCE
PANEL D
FIG. AI-21

[redacted]


TRANSLATIONAL FORCE
PANEL E
FIG. AI-22
PRESSURE (p.s.i)
TIME (sec.)
No. 5
No. 4
No. 2
60
50
40
30
20
10
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

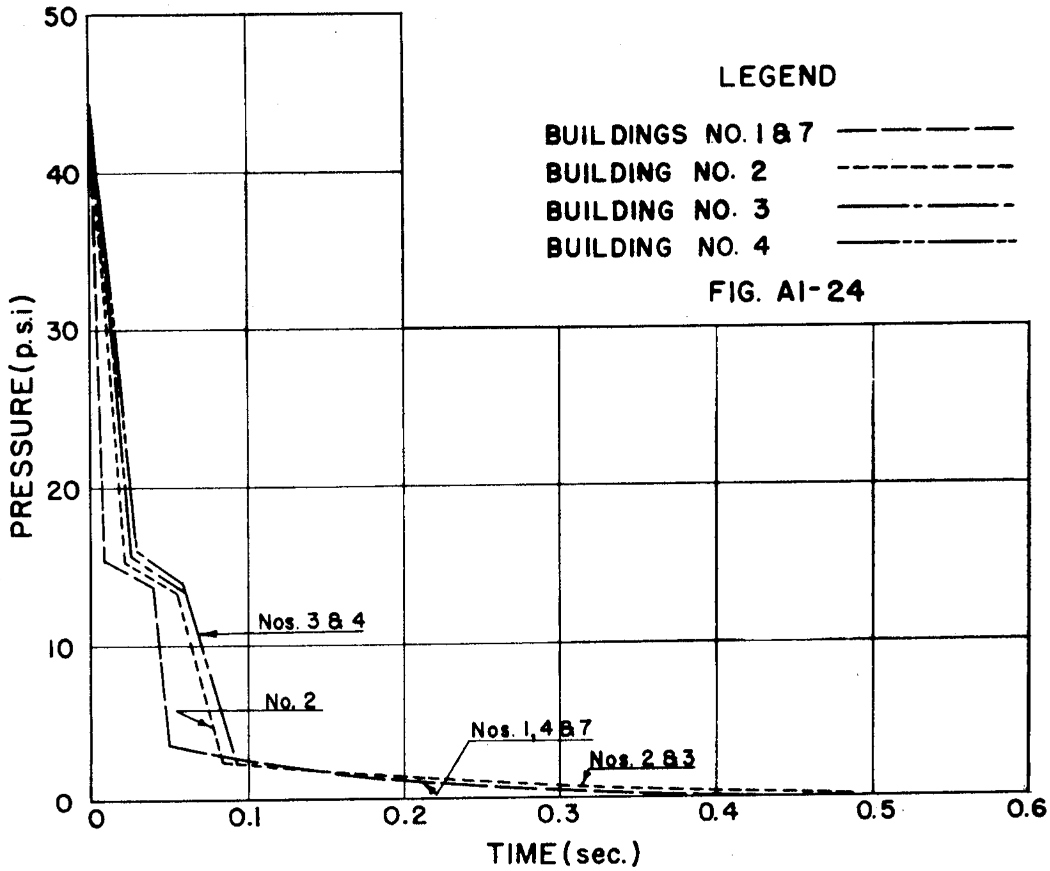
[redacted]



TRANSLATIONAL FORCE
PANEL F

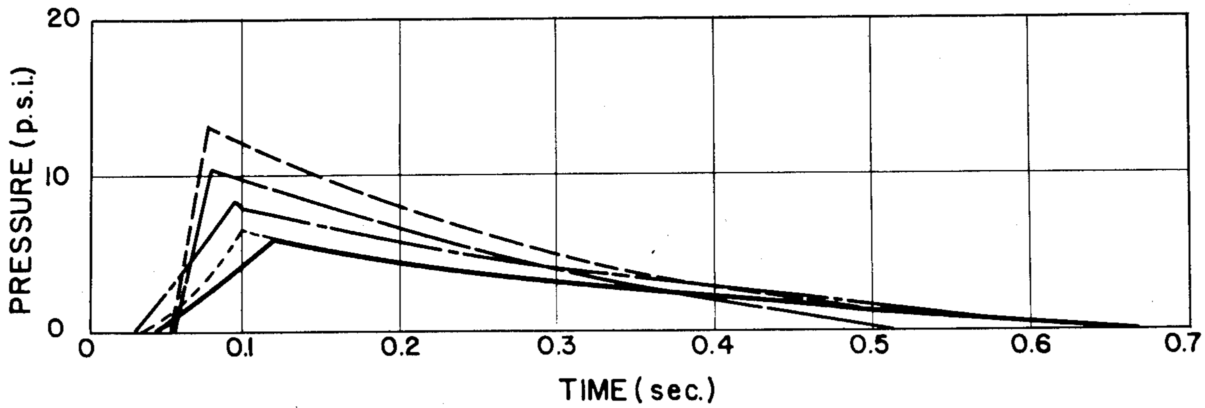
FIG. A1-23

[redacted]



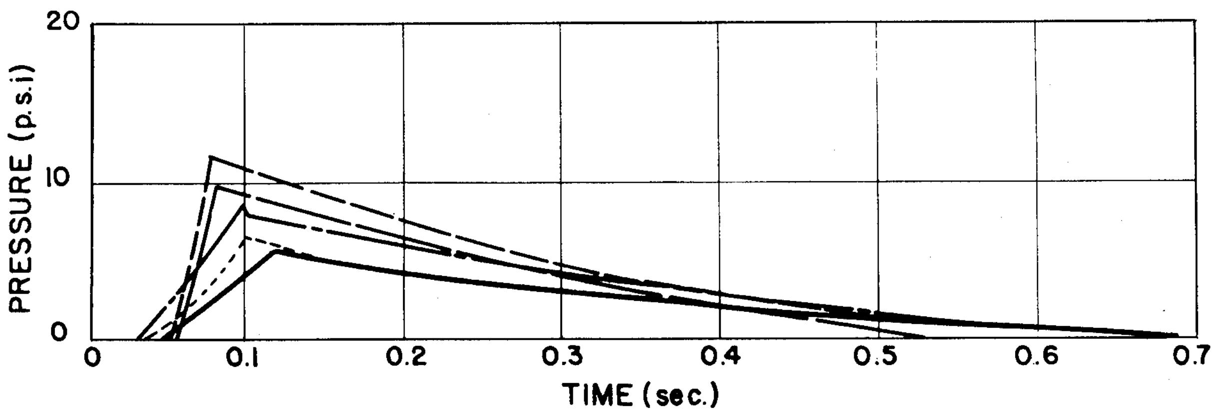
AVERAGE TRANSLATIONAL FORCE – ISSUE NO. 1
BUILDINGS 1 TO 4 AND 7

[redacted]



PRESSURE ON PANEL A-REAR WALL

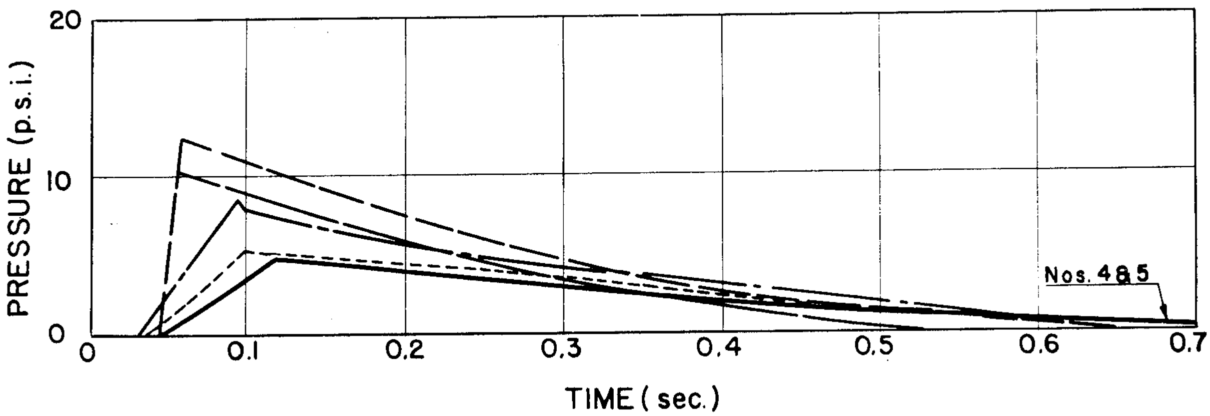
FIG. A1-25



PRESSURE ON PANEL A'-REAR WALL

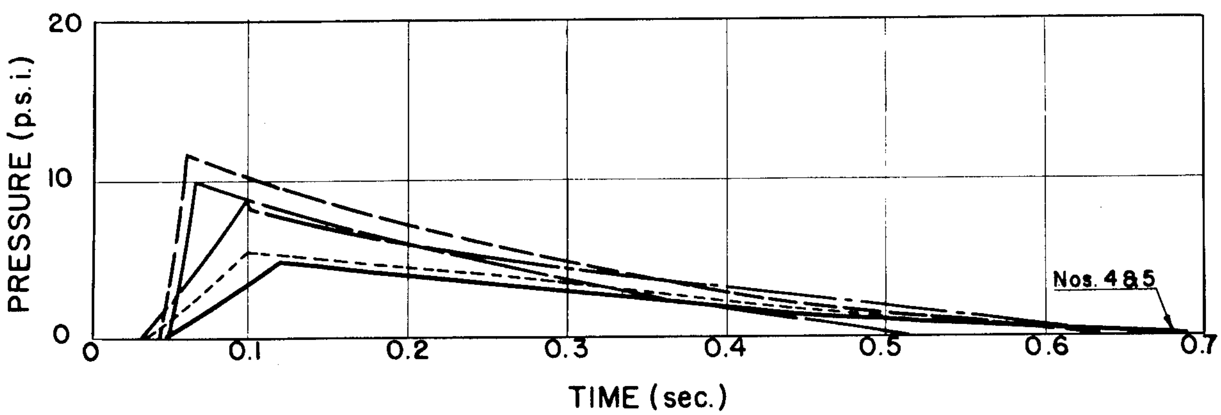
FIG. A1-26

[redacted]



PRESSURE ON PANEL B-REAR WALL

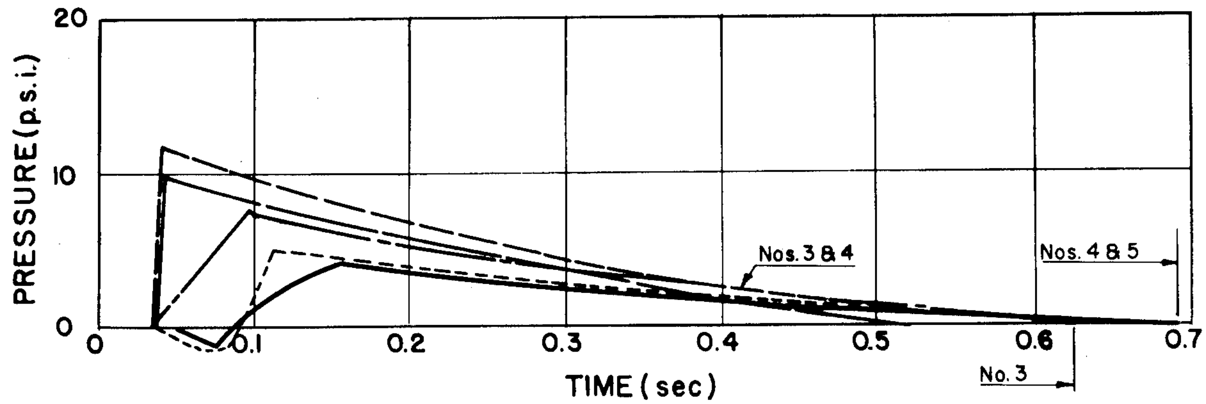
FIG. AI-27



PRESSURE ON PANEL B'-REAR WALL

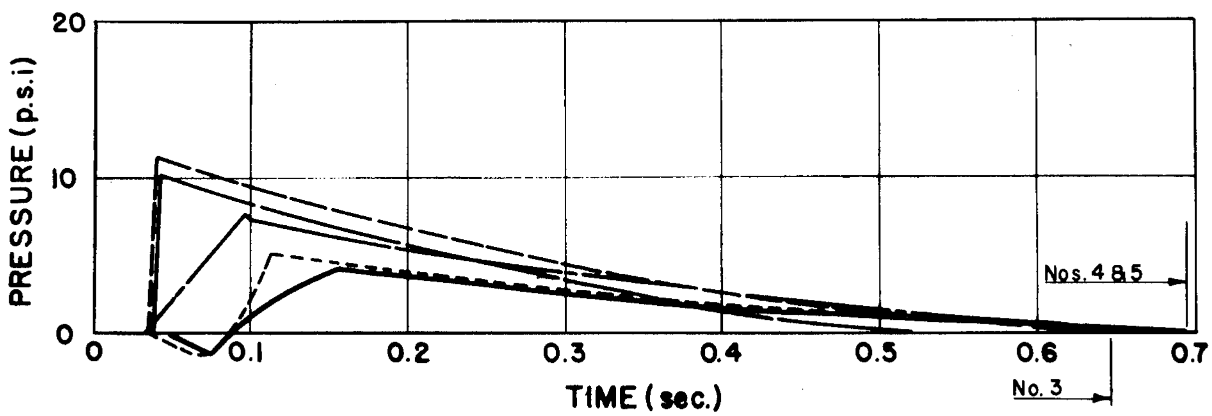
FIG. AI-28

[redacted]



PRESSURE ON PANEL C–REAR WALL

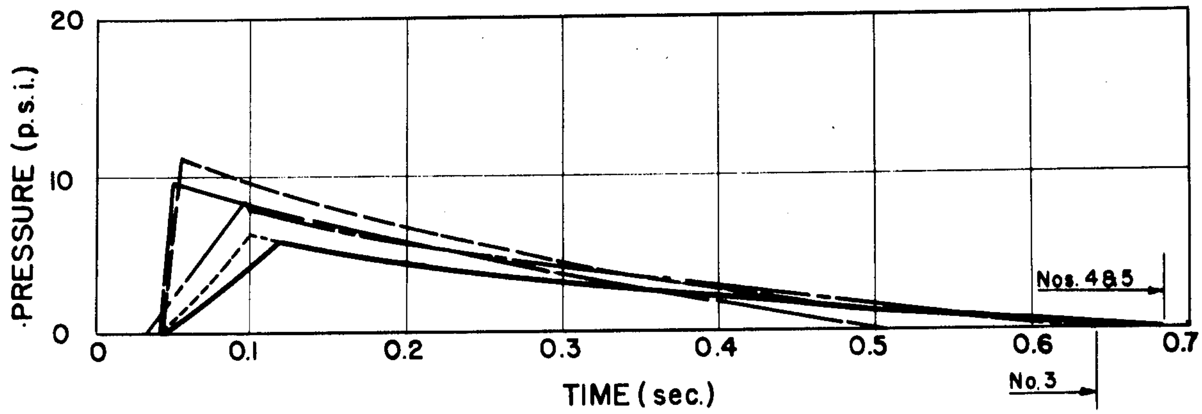
FIG. A1-29



PRESSURE ON PANEL C'–REAR WALL

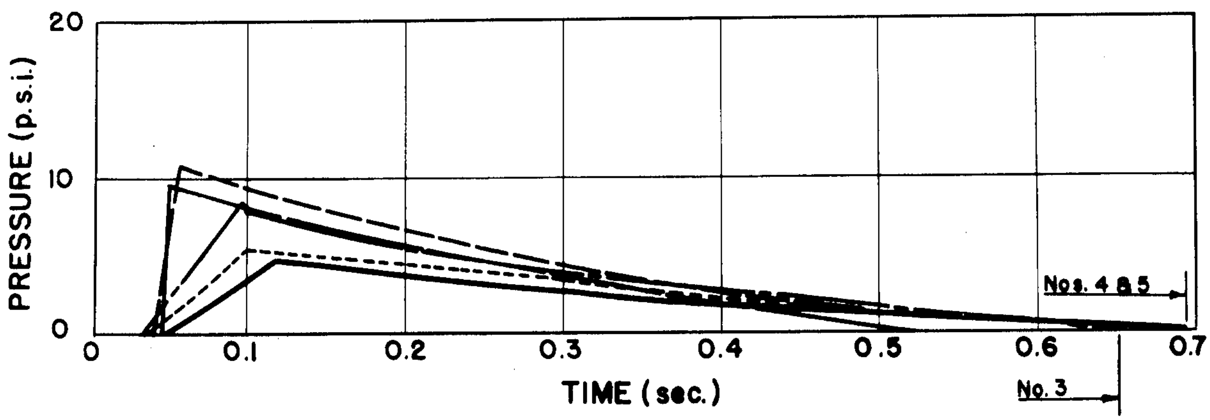
FIG. A1-30

[redacted]



PRESSURE ON PANEL D-REAR WALL

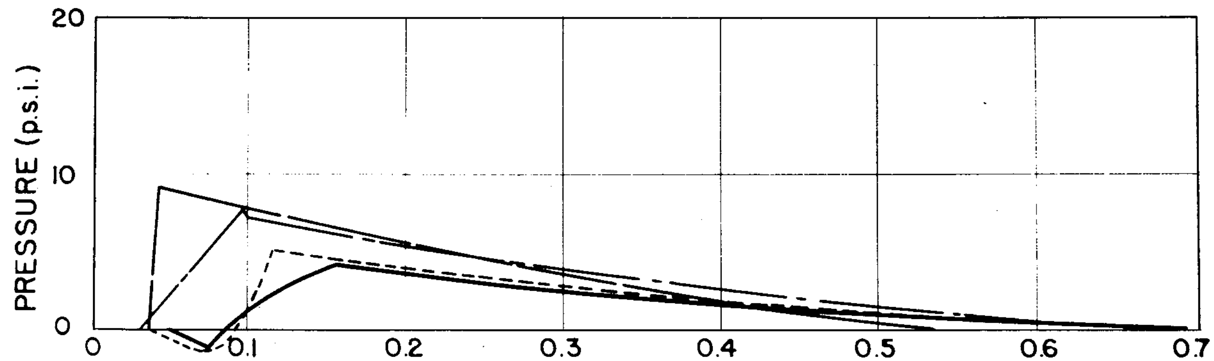
FIG. AI-31



PRESSURE ON PANEL E-REAR WALL

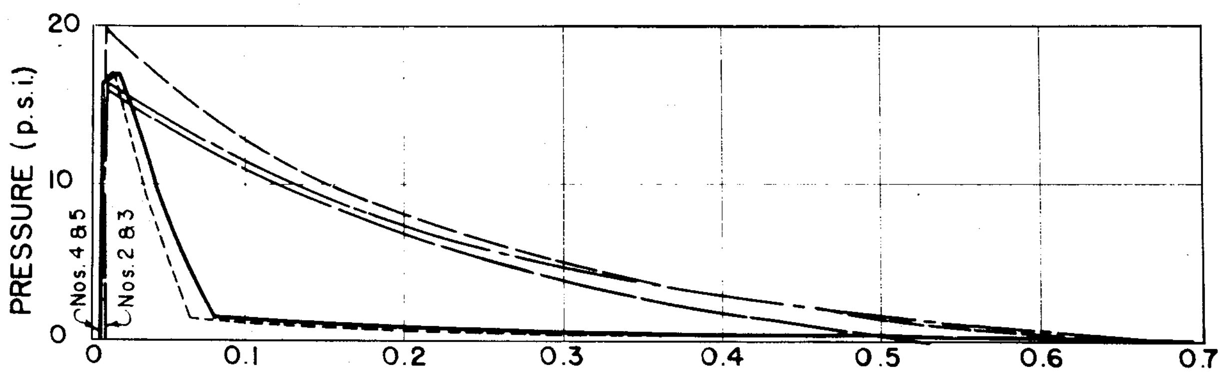
FIG. AI-32

[redacted]



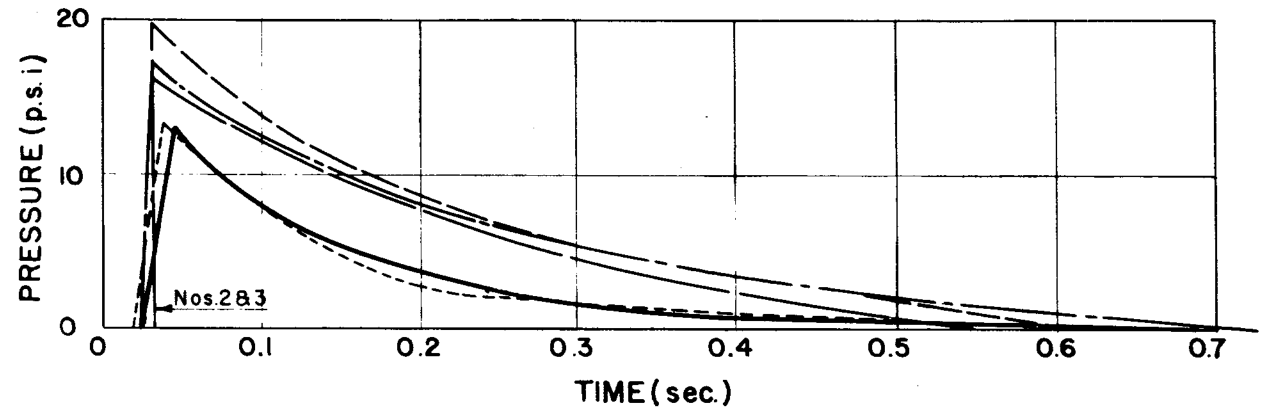
PRESSURE ON PANEL F-REAR WALL

FIG. AI-33



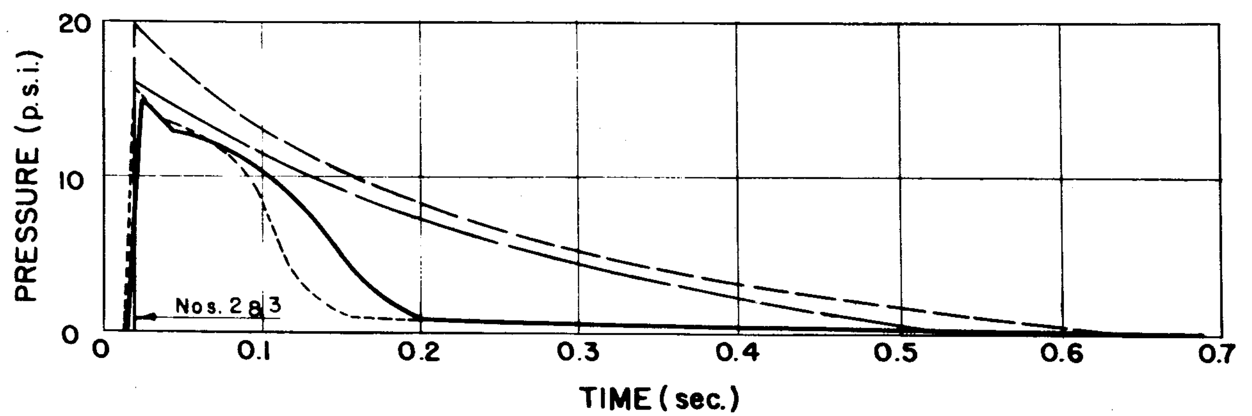
NET PRESSURE ON ROOF PANEL A-B FOR BUILDINGS 1 TO 4 AND 7

FIG. AI-34



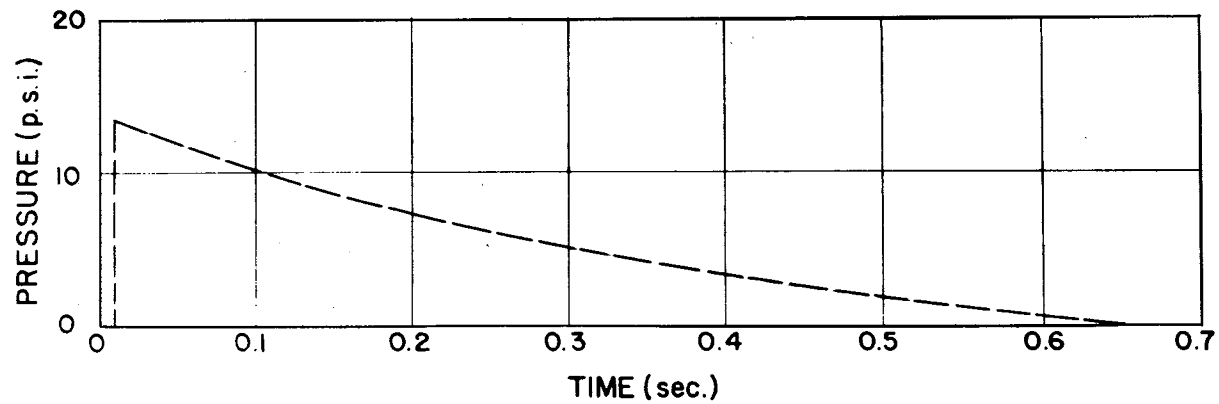
NET PRESSURE ON ROOF PANEL C-D FOR BUILDINGS 1 TO 4 AND 7

FIG. A1-35



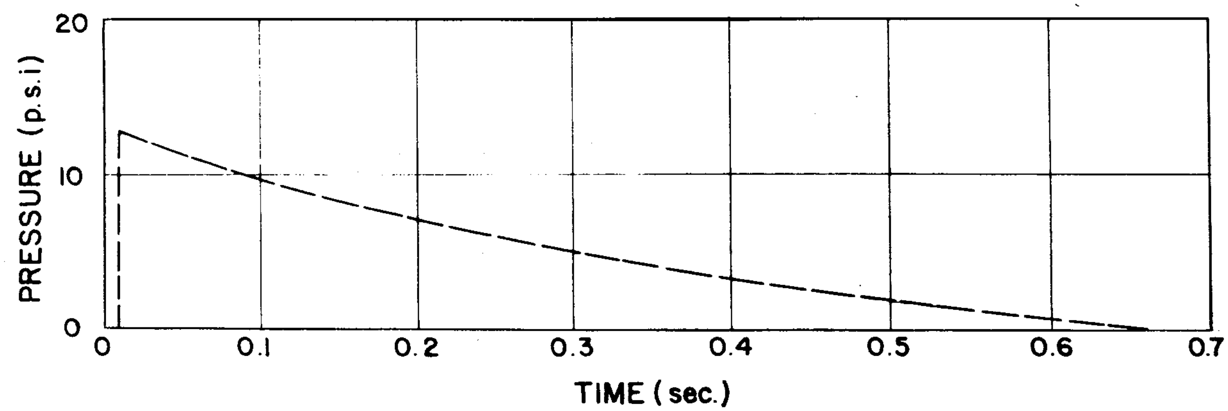
NET PRESSURE ON ROOF MIDPOINT FOR BUILDINGS 1 TO 4 AND 7

FIG. A1-36



PRESSURE ON SHEAR WALL BETWEEN BUILDINGS 4 & 5 AND BUILDINGS 6 & 7
FIRST FLOOR

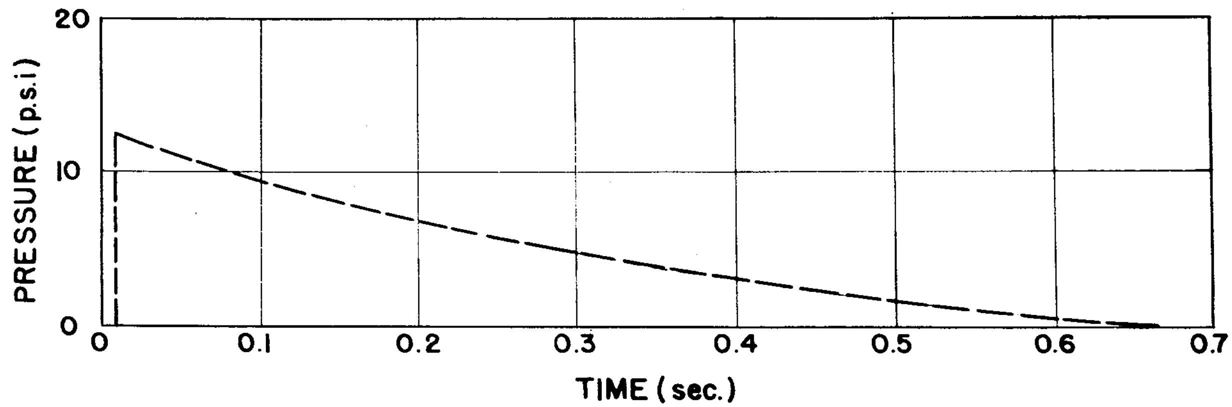
FIG. AI-37



PRESSURE ON SHEAR WALL BETWEEN BUILDINGS 4 & 5 AND BUILDINGS 6 & 7
SECOND FLOOR

FIG. AI-38

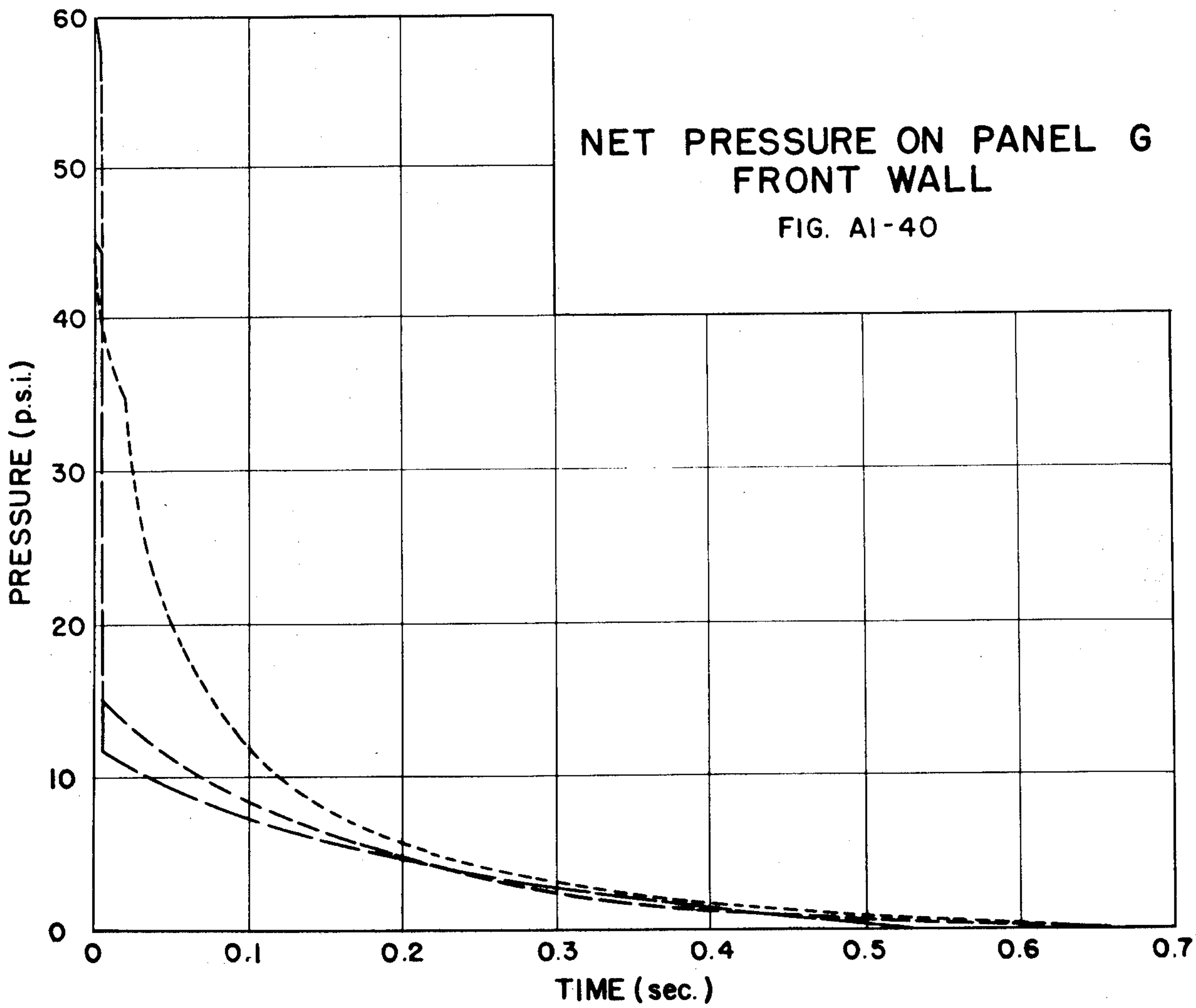
[redacted]



PRESSURE ON SHEAR WALL BETWEEN BUILDINGS 4 & 5 AND BUILDINGS 6 & 7
THIRD FLOOR

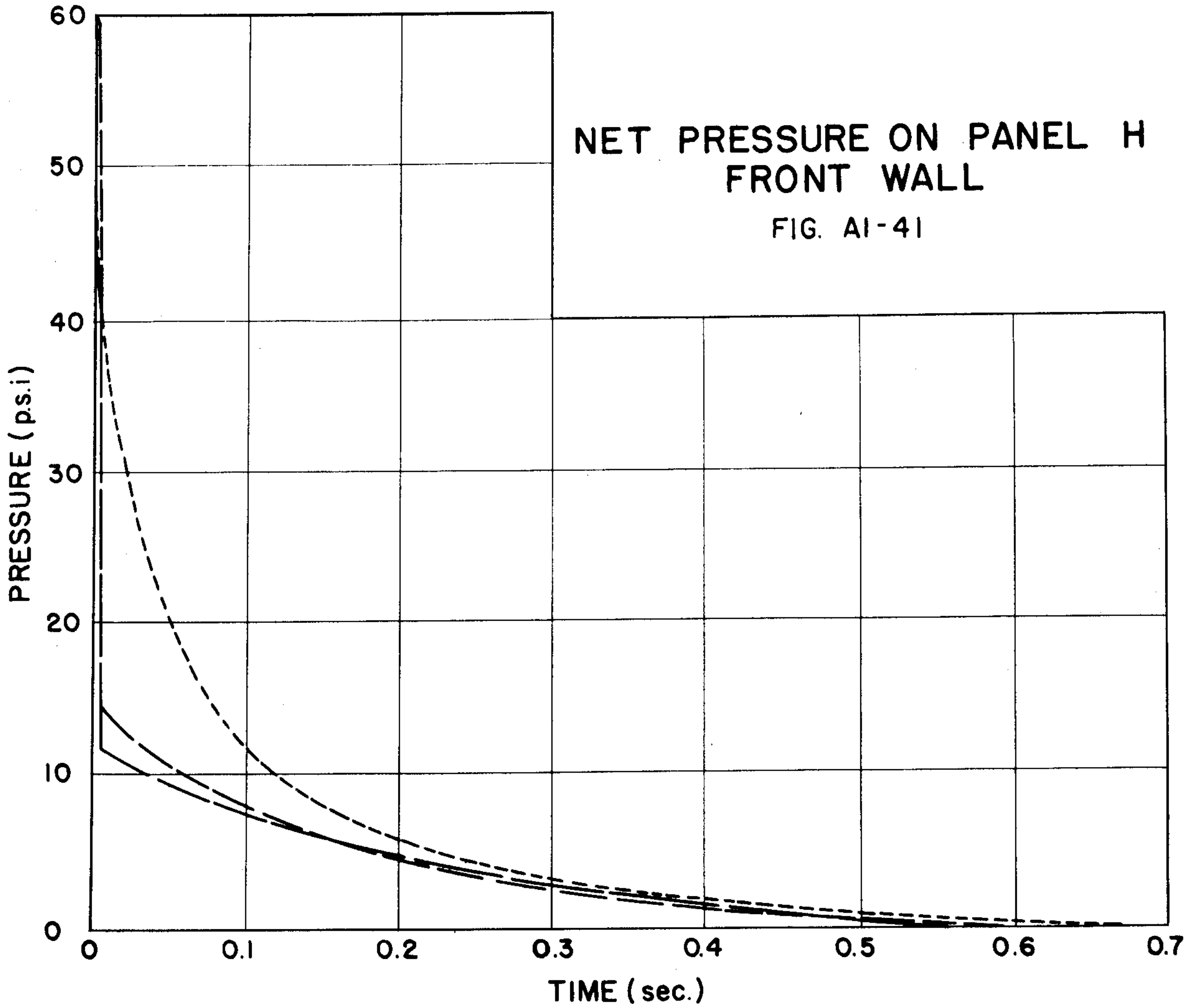
FIG. A1-39

[redacted]



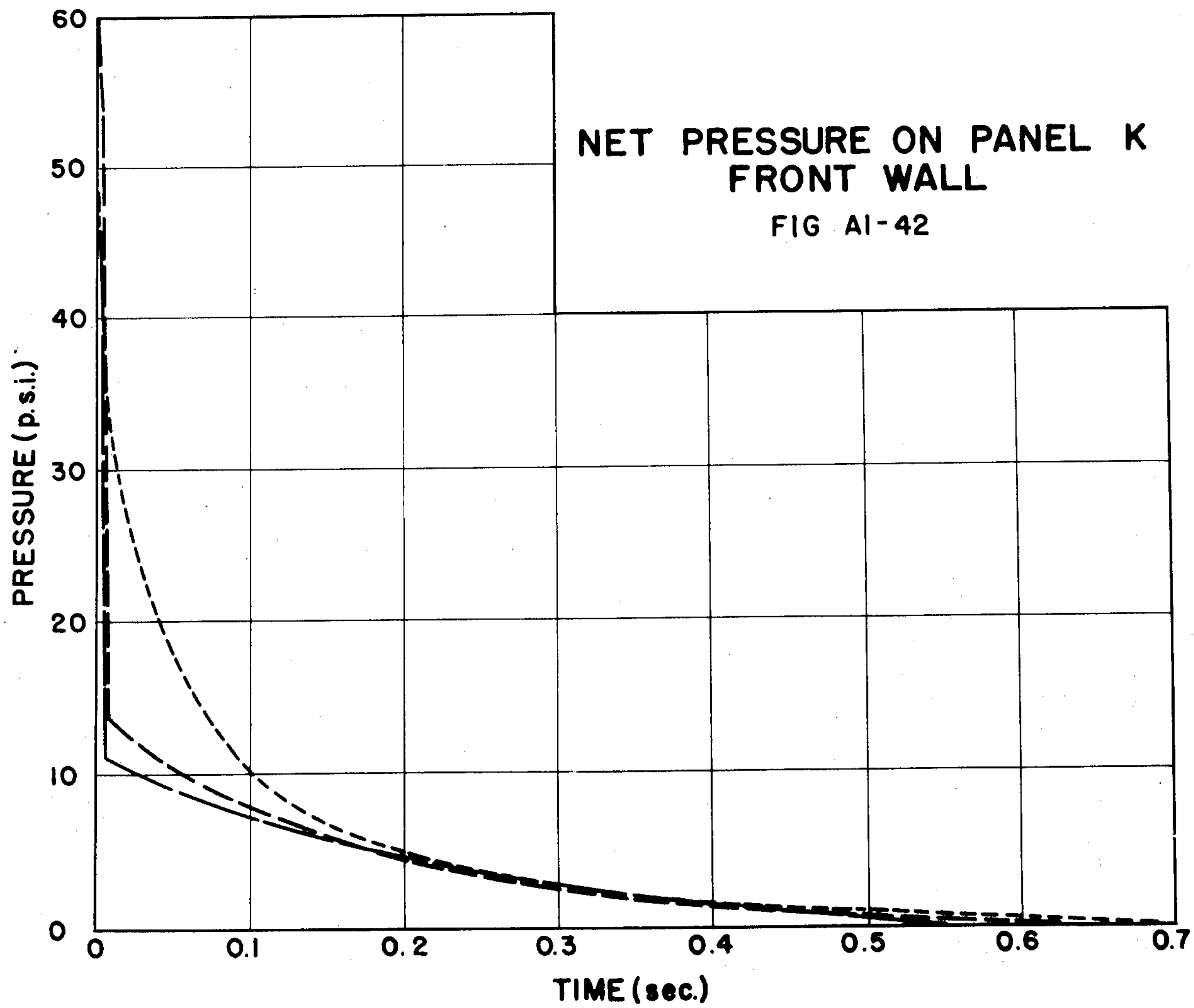
NET PRESSURE ON PANEL G
FRONT WALL
FIG. A1-40

[redacted]

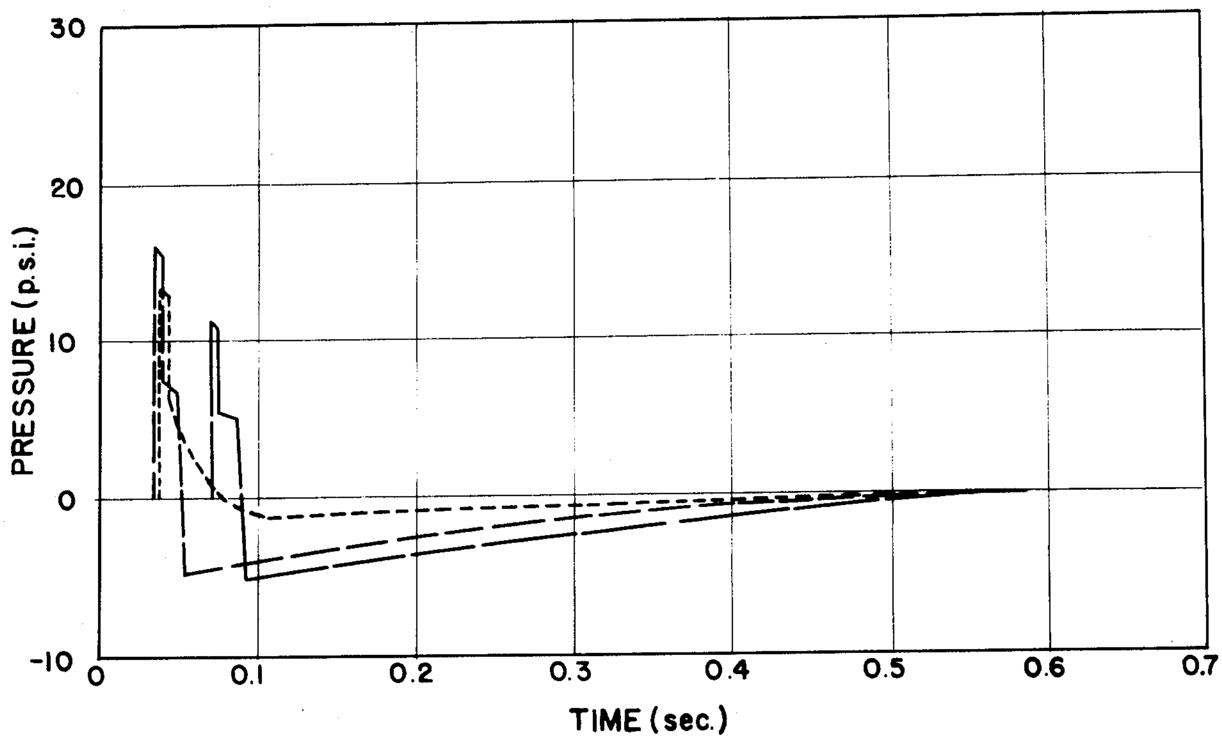


NET PRESSURE ON PANEL H
FRONT WALL
FIG. A1-41

[redacted]


NET PRESSURE ON PANEL K
FRONT WALL
FIG AI-42
PRESSURE (p.s.i.)
TIME (sec.)
0
10
20
30
40
50
60
0.1
0.2
0.3
0.4
0.5
0.6
0.7

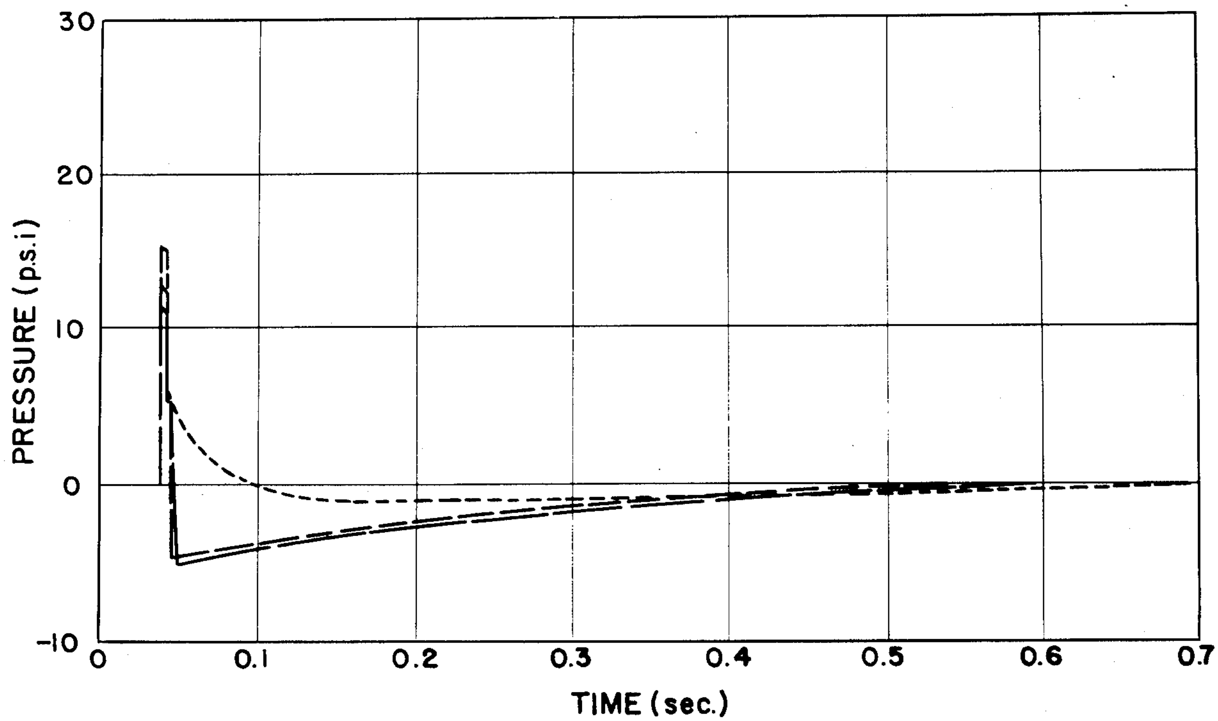
[redacted]



NET PRESSURE ON PANEL G
REAR WALL

FIG. A1-43

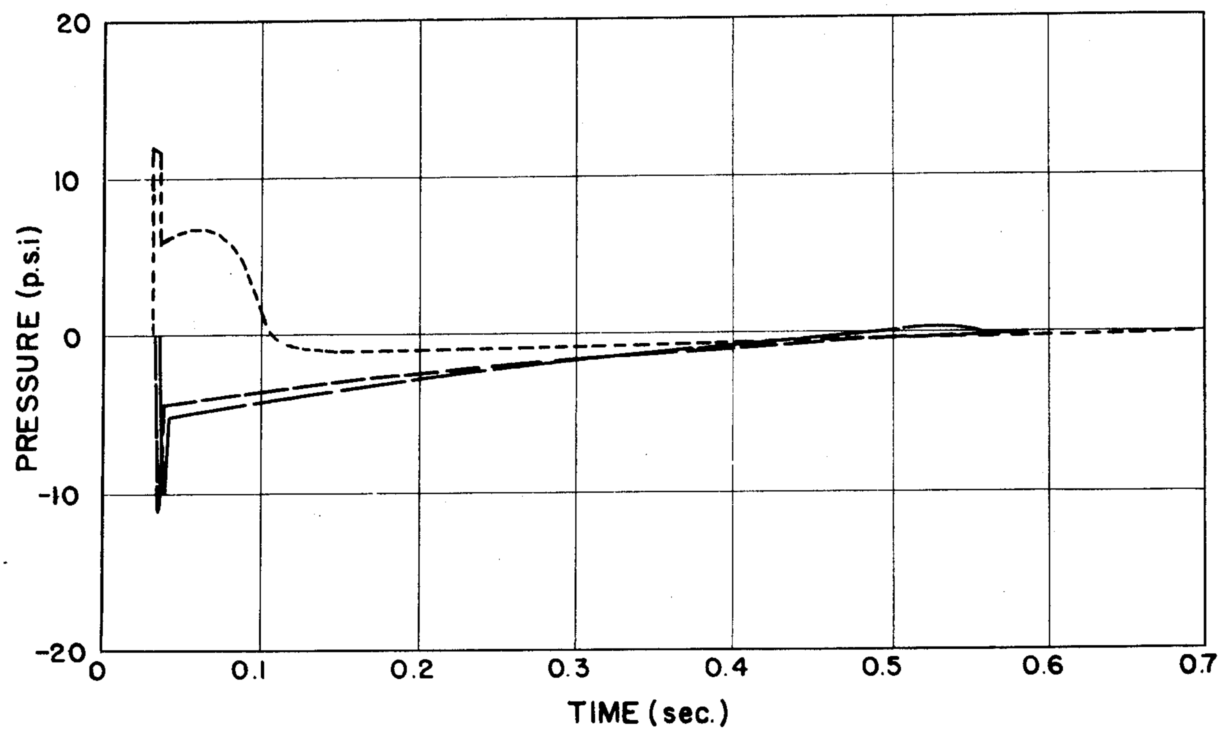
[redacted]



NET PRESSURE ON PANEL H
REAR WALL

FIG. A1-44

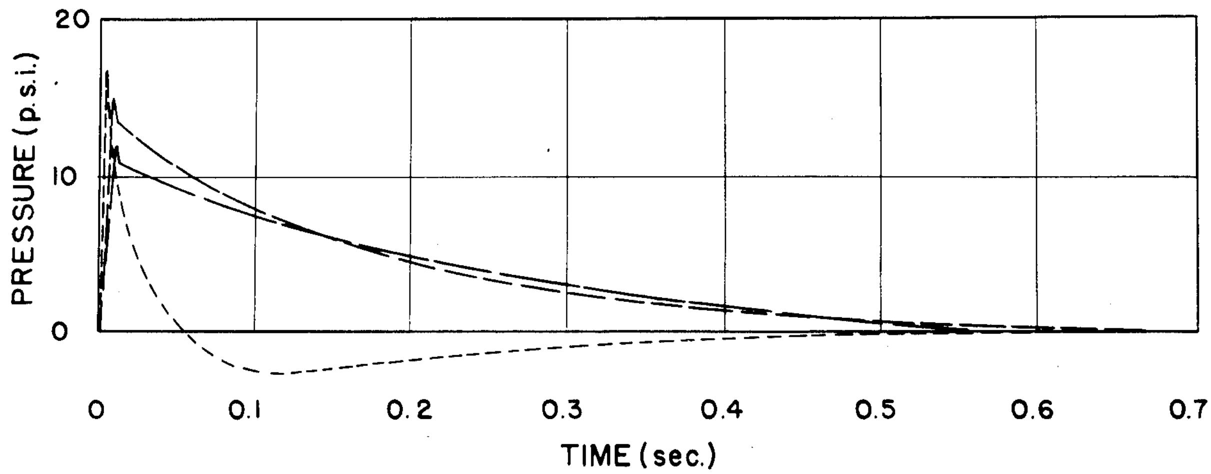
[redacted]



NET PRESSURE ON PANEL K
REAR WALL

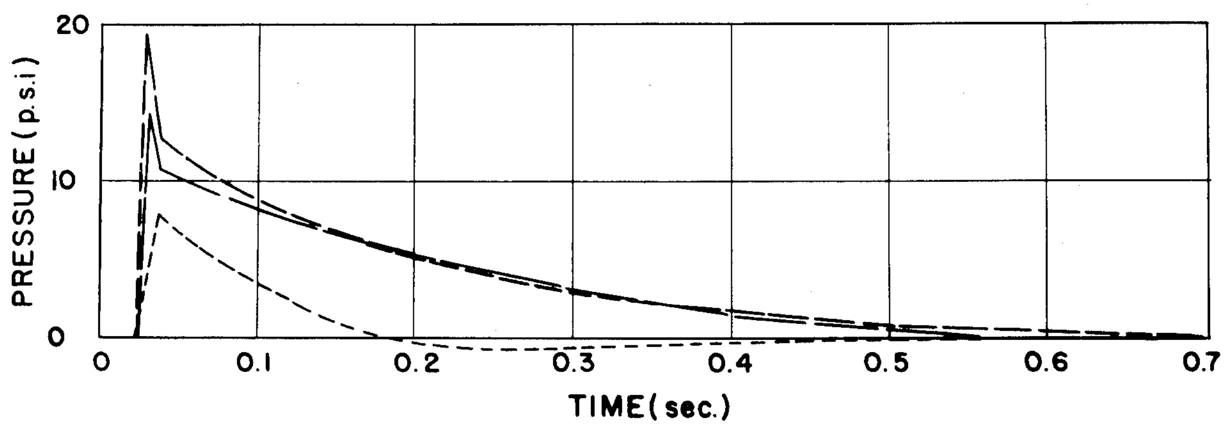
FIG. AI-45

[redacted]



NET PRESSURE ON ROOF PANEL A-B FOR BUILDINGS 5 & 6

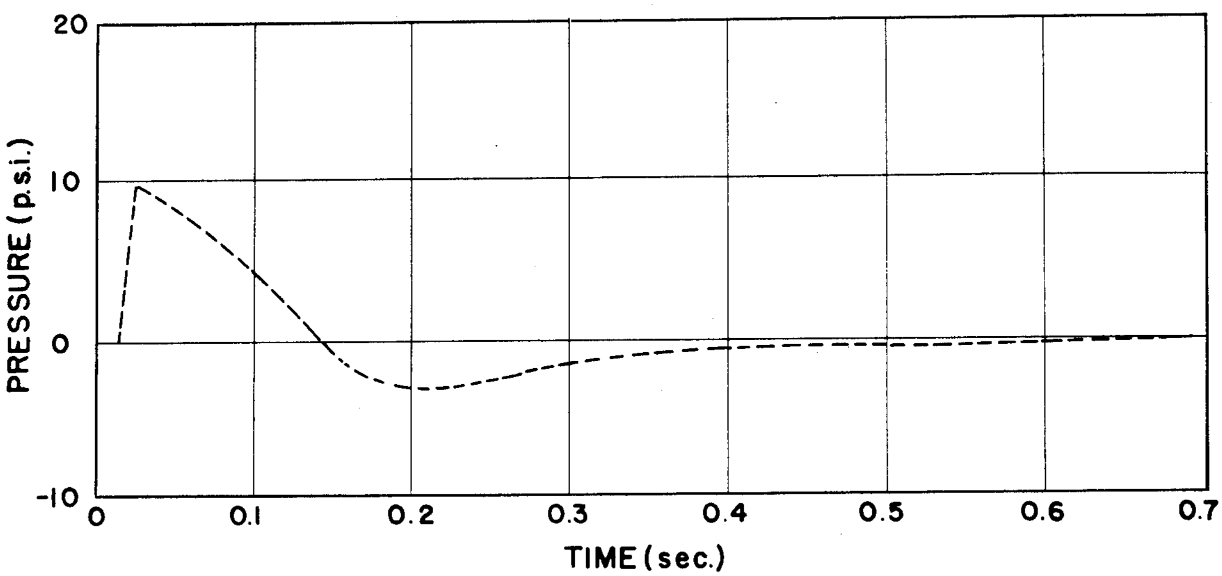
FIG. A1-46



NET PRESSURE ON ROOF PANEL C-D FOR BUILDINGS 5 & 6

FIG. A1-47

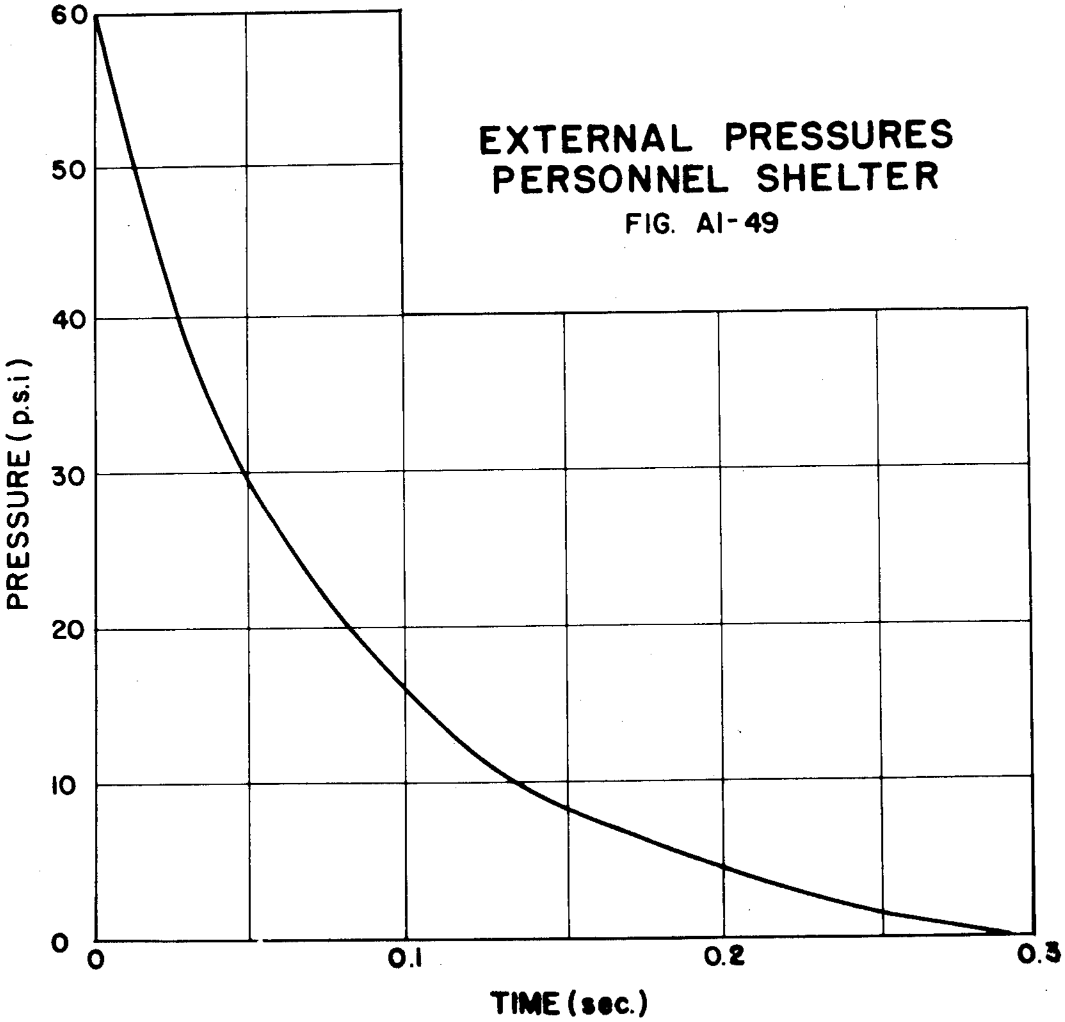
[redacted]



NET PRESSURE ON ROOF MIDPOINT FOR BUILDINGS 5 & 6

FIG. A1-48

[redacted]



EXTERNAL PRESSURES
PERSONNEL SHELTER
FIG. AI-49

[redacted]

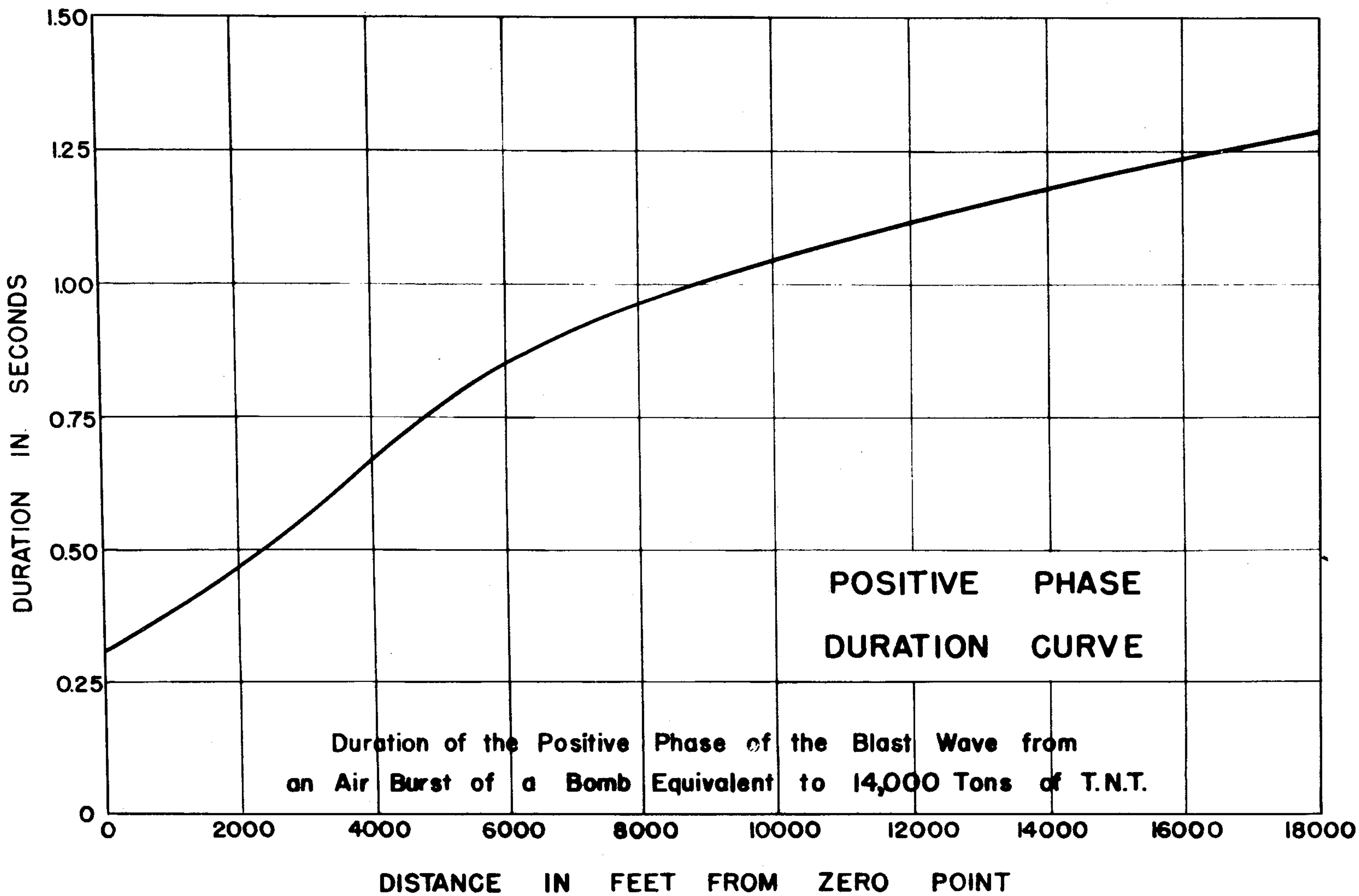


FIG. A1-50

[redacted]

APPENDIX 2

DESCRIPTION OF TEST STRUCTURES

CONTENTS

A2.1 Introduction

The multi-story building is designed and arranged to test different types of materials and framing on buildings with and without window openings. Besides a wide range in general types of framing and coverings, further detailed variations within the particular types are also provided. The members are intended to undergo different degrees of elastic and plastic deformation and include reinforced concrete members with the same ultimate strength but of different sizes and with different steel percentages, members with the same flexural strength but with varying shear and bond strengths, and several series of members having strengths graduated to cover the probable variation of the theoretical blast pressure. The tables of Appendix 3 summarize these design conditions.

The design methods used in this analysis are described in Part 2 and sample calculations are shown in Appendix 5. Therefore they will not be repeated in these sections.

The concrete strength is specified at 3000 p.s.i. and may be expected to be equal to or greater than this value. The steel strengths are specified as 32,000 p.s.i. and 40,000 p.s.i. for structural and intermediate grades respectively. However, as this is a minimum requirement, static yield strengths of 40,000 p.s.i. and 50,000 p.s.i. were used in the design of the members in accordance with actual average yield points obtained on numerous other projects. Ultimate dynamic strengths were then calculated for each design condition in accordance with the suggested procedure of section 2.5.2 and the expected rates of loading.

A2.2 General Arrangement of Test Structures

As described earlier, the test structures are composed of seven separate test buildings. Five of these buildings represent distinct framing types; the other two, while primarily functioning as closures to seal the

[redacted]

ends, also act to furnish numerous auxiliary tests for particular types of curtain walls and slabs which are not specifically included on the other five buildings. The buildings are so arranged with respect to each other that the end buildings and the center shear wall building automatically protect the remaining four sections from blast pressure penetration which would otherwise prevent the desired individual behavior.

A double layer of closely woven glass fabric covers the openings between the buildings and prevents the blast pressures from entering the interiors except through the window openings of some of the buildings. This fabric is folded to permit each of the seven buildings to move freely with respect to the others without destroying the seal.

A2.3 Detailed Description of Individual Buildings in the Test Structures

A2.3.1 Concrete Frame Buildings - Buildings No. 3 and 5

A. Introduction

Buildings No. 3 and 5 are reinforced concrete frame structures similar to each other in detail except for the presence of windows in Building No. 5. These buildings are more or less typical of conventional construction except for the greater strength and certain detail modifications necessary to resist the intense shock of the blast loads. The strength of the frames is the same for both buildings although they are subjected to different blast loadings due to the windows in Building No. 5. The total resistance to lateral deflection, however, is somewhat less for Building No. 5 than Building No. 3 because the strength of its walls, which contribute to the resistance, is reduced by the window openings.

The total translational force applied on Building No. 5 is less per unit area and acts on less area than on Building No. 3. Figure A2.3.1-1 shows the total translational force on each building.

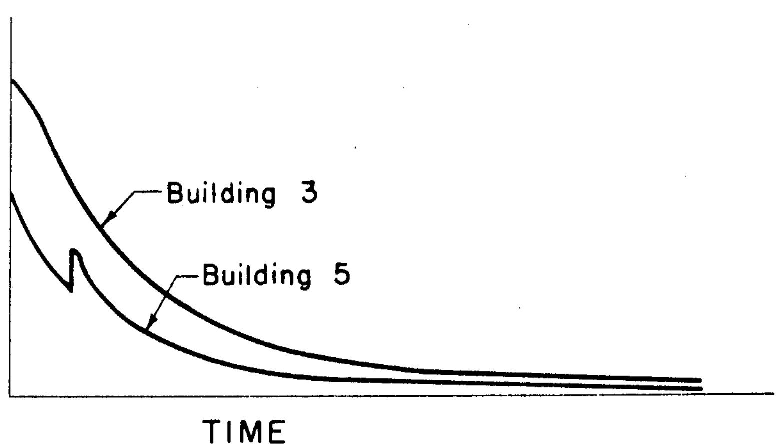


FIG. A 2.3.1-1

The difference in the ratio of the applied loads and the resisting strengths of the two buildings helps assure that at least one building will deflect within the desired range despite the possibility of variations in the blast pressure. If the theoretical pressures are substantially correct, then two sets of readings will be available in which both buildings will have plastic deformations but will show different degrees of damage. The latter information is desirable for use in the detailed post-test examination of the proposed design procedures.

B. General Arrangement

The reinforced concrete frame structures consist of three-story sections each supported by two lines of bents framed in the direction of the blast as shown by Drawings 60-09-06, Sheets 1 and 9 of Appendix 7.

The two frame bents are each composed of heavy floor girders, running from the front to the rear wall, supported by two interior columns and an exterior wall column at each end. The exterior columns are set back to prevent contact with the wall at its maximum anticipated deflected position.

The front and rear walls are continuous members extending from the foundation to the roof level. These walls are tied to the floor slabs at each floor level.

The floors and roof consist of one way slabs spanning between and cantilevering over the floor and roof girders which are integral parts of the frame bents.

The foundations for these buildings consist of heavy continuous reinforced concrete members continuous from front to rear under each frame line. The ends of these strip footings are joined by connecting members at each end which receive and restrain the bottom ends of the front and rear walls. A concrete slab on the subgrade is provided at the ground floor level.

C. Details of Framing and Design

1. Foundations

The foundation structure is shown in detail on Sheet 9 of Appendix 7. The continuous members under the front and rear walls are 4 ft. by 4 ft. in section and they extend $1\frac{1}{2}$ ft. beyond the face of the front and rear walls.

The continuous strip footings running from the front to the rear are reinforced concrete members 4 ft. deep by $3\frac{1}{2}$ ft. wide. These strips carry the axial load, shear, and moment of each column in the bents above in addition to the thrust, shear, and torque of the wall footings. The footings are designed for a number of possible loads including the condition of maximum vertical load in the columns, usually occurring early in the loading when the roof

[redacted]

load is a maximum and the column moments are low; and the condition of maximum moment occurring when the column moments and column shears are at a maximum value, the roof load and hence vertical loads acting at a reduced value at this time.

These footings were designed as members under combined bending and direct stress, using the plastic theory to estimate the strength. The stresses will not exceed the elastic limit at any time, the purpose being to confine the plastic action and points of high strain to the instrumented and visible frames above.

2. <u>Second and Third Floor Slabs</u>

The second and third floors are framed by $4\frac{1}{2}$ inch reinforced concrete slabs which span 10'-6" between centers of the floor girders and cantilever outward 4'-$3\frac{1}{2}$" on each side, as shown by Section c-c, Sheet 9 of Appendix 7.

These slabs are designed for dead load plus a 150 p.s.f. live load in accordance with the American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318-47). In addition the floor slabs are designed to resist the horizontal shearing stresses in the plane of the slab due to the wall reactions. The slabs were also checked against the dynamic action of the dead and live loads due to the vertical motion of the bents. The floors are one-way slabs spanning between the floor beams except at the front and rear walls, where they are supported by the walls as well as the floor beams.

3. <u>Roof Slab</u>

The concrete roof slab is framed in a manner similar to the typical floor slabs except that the conventional design for dead and live loads is replaced by a plastic limit design for dead load plus the dynamic roof loads due to the blast pressure. The roof slab is 9 inches in depth and was originally designed for a deflection of $L/32$ under pressures increased 35% in intensity over the theoretical pressure curve. Later revisions in the pressure loading and a change in the position of the building reducedthe expected deflections to near the elastic range.

This reduction in the deflection of the roof slabs is probably not too important as a sufficient number of other test panels are available to check the design procedures for deflections in the plastic range and it is advantageous to make sure that the desired amounts of frame deformation will occur before the coverings fail.

The roof slab was also designed as a one-way member except in the vicinity of the front and rear walls. The methods of section 2.5 were used directly in this solution.

4. Slab on Subgrade

In order to provide a clean working surface, a lightly reinforced 4 inch slab on subgrade is provided at the first floor level.

5. Front and Rear Walls

The front and rear faces of sections 3 and 5 are covered by solid monolithic concrete walls except for Building 5, which has window openings in each story, front and rear. These walls are indicated and detailed on Sheets, 1, 9, and 11 of Appendix 7.

To provide adequate room for the four layers of reinforcing bars in each wall (vertical and horizontal, both faces) and because of the beneficial effects of high mass values, all vertical walls are one foot in thickness. Special web reinforcement is provided where the shearing stresses are excessive.

The front walls of Building 3 were designed to resist the blast pressures at a maximum local deflection of L/32 in each story. They were further investigated for the effect of wall participation in frame action as is described in Section 2.4.5.

The rear wall is of the same thickness and has substantially the same reinforcement as the front wall. These were so designed because the direction of the blast may be unknown in the future. Because the blast pressure is less on the rear face, the over-reinforced rear wall panels will carry the local blast pressure near or within the elastic range though the stresses will reach the yield point value under the large horizontal motion or sidesway of the frames.

Besides the framing adopted for the test structure several other framing types were examined and of these, two were studied in detail.

In the first type the vertical wall was hinged at the connection to the footing. This condition permitted the use of lighter wall footings and reduced the effective participation of the walls in frame action. However as the required strength of both the walls and the frame was increased, the method was rejected.

In the second type the wall was fixed at each floor level by means of spandrel beams framing between the floor girders. These beams were sufficiently stiff to practically fix the wall at each floor level. This framing resulted in a shorter wall span because of the depth of the added spandrel beam; the wall columns were eliminated, and more resistance was developed against lateral sidesway. These savings, however, were largely balanced by the cost of the added spandrel beams and the somewhat increased strength necessary in the floor girders. Furthermore, the resistance developed by the upper floors is invariably too great, leading to difficulties in proportioning the relative floor strengths.

6. Columns and Girders

As described previously, the main framing consists of two lines of bents or frames, each frame consisting of two interior and two exterior wall columns.

The wall columns were designed to be approximately one-half the strength of the interior columns as there is only the single outer girder to resist the total column moment from above and below, whereas at the inner columns the sum of the column moments from the upper and lower columns is distributed in two directions.

The columns are rectangular in shape and are designed as tied members though every bar is tied at close intervals in the vicinity of the expected plastic hinges. This design was adopted because the moments are large in comparison to the axial stress. If, in other designs, the axial load becomes important enough to indicate the possibility of a primary compression failure in the columns, the use of spirally reinforced columns would be preferable even at the price of lowered efficiency against lateral bending. Spiral columns are able to undergo large plastic strains in the concrete during the early high-intensity axial loading.

The floor girders are designed to remain within the elastic range during the plastic yield of the column. While it is possible to have the plastic joints in either the columns or the girder, the difference in design strength will not be appreciably changed by either method as the weakest member will govern the resistance developed against sidesway. The decision to provide the plastic hinges in the column rather than in the girder was made after considering that the general damage will probably be less if the cracking occurs in the column rather than in the floor system, that the columns are generally smaller and better able to undergo large strains than the girders, and that the behavior and developed strength of the girders is more difficult to predict due to the combination of the effects of local floor loads, sidesway and the varying stiffness and strength of the floor system as a whole.

The frames were designed in accordance with the procedures described in section 2.4.5 and as shown by example in Appendix 5.

[redacted]

A2.3.2 Steel Frame Buildings - Buildings No. 2 and 6

A. Introduction

Buildings No. 2 and No. 6 are intended to perform the same function for steel framing that Buildings No. 3 and No. 5 do for reinforced concrete framing. They too have frames of equal strength and one section has windows while the other does not. These buildings are typical of conventional building framing except that the details are modified by the intensity of the loads.

B. General Arrangement

Again, like the concrete buildings, the steel framed buildings are three-story structures. Each section is 19'-1" in width with two frames or bents each consisting of two lines of rigid steel frames, spaced 10'-6" apart as shown on the plans and details of Drawing 60-09-06, Sheets 1, 6, and 7 in Appendix 7. The floor and roof are composed of $4\frac{1}{2}$" and 8" one-way slabs respectively, each slab spanning between and cantilevering over the steel floor girders with a slab reinforcing arrangement similar to that in Buildings No. 3 and No. 5. The walls consist of corrugated metal siding which, in most cases, is supported on vertical girts which span from floor to floor. The girts are discontinuous at each floor level.

C. Detailed Framing and Design

1. Foundations

The foundations are solid 5 feet deep mat foundations projecting 8 feet in front of and 1'-6" in back of the front and rear wall faces respectively. Mat foundations were used to provide the weight necessary to resist overturning or rotation under a load approximately 135% of the given theoretical blast pressures. This foundation, like those of Buildings No. 3 and No. 5, is designed for a number of loading conditions ranging from the initial condition of maximum vertical column loads to the condition of maximum moments and shears with minimum thrusts. The direct stress produced by the horizontal reaction of the lower end of the front wall plus the column shears has a great effect on reducing the quantity of footing steel required by flexure. The force resisting sliding at any point on the base of the footing is assumed to be proportional to the vertical pressure acting at that point. The footing projecting beyond the face of the front walls receives vertical blast pressures which serve to stabilize the building against overturning. This pressure also provides an end moment and vertical downward force which is effective in neutralizing the otherwise high bending moments in the footing due to column axial loads, shears, and moments.

[redacted]

The footings are designed to remain elastic under the maximum loading conditions.

2. Floor Slabs

The floor slabs are connected to the steel floor girders by means of shear clips formed by light 3" channels. The shear clips function partly to anchor the floor slabs to the steel beams and more importantly to transmit blast pressure loads from the walls to the frame. The shear clips also act to form a composite section of the beams and slab thus stiffening the girder and reducing the elastic girder deflections. The floor slabs are designed for dead load, a 150 p.s.f. live load, and shear stresses in the plane of the slab. The slab is designed for the dead and live loads in accordance with the conventional regulations of the American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318-47) except where controlled by the stresses produced by the lateral forces.

The intermediate floors are designed assuming that the blast pressures either do not occur on these surfaces, as is the case for Building No. 2 which is completely sealed, or that the pressures are equal on the top and bottom surfaces as in Building No. 6.

3. Roof Slabs

The general framing arrangement and loading conditions are the same as for the concrete buildings except that the slab action is assumed one-way throughout. Shear clips are used between the roof girders and the roof slab.

4. Walls

The walls consist of protected corrugated metal supported by wide flange beams. These beams, or girts, are discontinuous at each floor and the positive reactions are supplied by direct bearing against the floor slab. Negative reactions and rebound at each end of each girt are resisted by two bolts embedded in the floor slab. The corrugated siding and girts are designed for plastic deflection at the front face of the windowless building but are largely elastic in action on all other sides and in Building No. 6. The front and rear walls of these buildings are not identical in strength, the rear walls being designed for the smaller theoretical blast loads than the front. The front and rear walls of the steel building, contrary to the concrete wall behavior, do not contribute to the frame action and hence equal strengths for these members would be highly expensive without adding to the information to be gained in the test program. Details of the wall are shown on Sheet 8 of Appendix 7.

[redacted]

5. Columns and Girders

The original design contemplated using two member sizes for the three floors of columns with column splices near the midheight of the second and third floors. As it was impossible to use this arrangement under the final revised pressure loads and still maintain a fairly uniform floor to floor deflection, the framing was changed to sections which change size at each floor as shown on Sheet 7 of Appendix 7.

The original designs also used riveted split beams for the girder to column connections. Because of difficulties in developing the required strengths, these connections were changed to short stub sections welded to the columns. These girder stubs furnish sufficient room to develop the required rivet splice for the lighter center section of the girder. The connections are all designed for more conservative stresses than are used for the rolled sections, the welded joints being full groove welds with double the static yield point of the base metal and with equal ductility. The ultimate rivet loads for design purposes are limited to double the conventional allowable rivet loads.

All columns and girders are designed in accordance with methods described in section 2.4.5 C.

The roof columns are field spliced to short stubs welded to the roof girders. As shown by the drawing, all joints are shop welded while all the field splices are riveted.

A2.3.3 Shear Wall Building - Building No. 4

A. Introduction

Resistance to lateral deflection by use of shear walls, has been widely used on numerous buildings including many structures designed to resist earthquakes. In this type of building, an extremely rigid path is provided for transferring the lateral blast loads from each floor level to the foundations. The front wall reactions at each floor level are transmitted through the floor slabs to the shear walls and through the shear walls to the foundations. The system of interior columns and girders is not directly utilized for lateral resistance and therefore damage resulting from the blast is expected to be confined principally to the rigid shell formed by the front, rear, and shear walls. Relatively heavy damage may therefore be sustained by the exterior walls without harming the contents or impairing the utility of the building.

B. General Arrangement

The shear wall building is a completely enclosed three-story building of monolithic reinforced concrete, 36 feet high, 53 feet wide, and 52 feet deep in the direction of the blast. The interior is supported by two rigid frames each of which is framed in the direction of the blast and consists of four columns fixed at the base and continuous with floor and roof girders. The frames are spaced between the side walls to divide the building into three equal bays. The front, rear, and side walls are of reinforced concrete one foot thick as shown by Drawing 60-09-06, Sheets 1 and 10 of Appendix 7.

The exterior columns are set back from the walls to prevent contact with the front and rear walls at their maximum anticipated deflected positions. Although the front and rear walls if undamaged could carry the exterior column loads, these columns, placed away from the shell provide a boundary separating the front wall from the interior framing, and although there may be some damage in the front exterior bay, it is not expected to be serious.

The front and rear walls are continuous members which extend from the foundation to the roof level and are tied to the floor slabs at each floor level. They are designed as one-way slabs between floor levels except at the shear wall ends where an allowance was made for two-way action. During the rotation of the structure they act as vertical beams to transfer the vertical loads from the adjacent roof and floor surfaces, the front footing and the strap footings to the shear walls.

The roof is designed as a one-way slab spanning between the shear walls and the roof girders except at the front and rear wall

ends where they are designed as two-way slabs.

The floors are designed as one-way slabs, spanning between the shear walls and the floor girders, by conventional design as outlined in the American Concrete Institute Building Code (25). In addition they are analyzed to provide for the transfer of the lateral loads from the front wall to the shear walls.

The front and rear walls, the shear walls, and the floor and roof slabs have been made as monolithic as possible in order to have the building act as a unit.

The foundations for this building consist of reinforced concrete strap footings continuous from front to rear under each of the interior frames and conventional wall footings under the shear walls. These strap footings are monolithic with and restrain members under the front and rear walls which in turn restrain the bottom ends of these walls. A concrete slab on the subgrade is provided at the ground floor level. This slab has been made monolithic with the footings and provides a clean working surface as well as an efficient method of tying the foundations together.

This building is located between the concrete frame building without windows and the concrete frame building with windows, Buildings No. 3 and 5. The joints between the shear wall building and the adjacent concrete frame buildings are sealed against the blast pressure as explained in section A2.2.

Covered doorways through the shear walls provide a passage way through Building No. 4 at each floor level, and access holes are provided in each floor slab.

C. Details of Framing and Design

1. Foundations

The foundation structure is shown in detail on Drawing 60-09-06, Sheet 10 of Appendix 7. The continuous members under the front and rear walls are 6'-0" wide by 2'-0" deep and extend 4'-0" beyond the face of the front wall and 2'-6" beyond the face of the rear wall. The front wall footing, which acts as a beam in torsion between the front-to-rear footings, serves to supply the end moments at the base of the front wall, and also carries the vertical load from reflected front wall pressures acting on the 4 foot projection beyond the face of the front wall. The projection was provided specifically to utilize this vertical load to help resist the overturning forces. The rear wall footing was designed to provide the required bearing area at the rear wall. The magnitude of this reaction is discussed in subsection 4 below. In the course of the foundation analysis it was found that the magnitude of the horizontal load on the rear wall footing due to frictional resistance of

(25) American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-47)

[redacted]

the soil to lateral movement of the structure made it impractical to consider the transfer of this force as a function of the rear wall footing acting as a beam between the shear walls. It was therefore decided to provide a suitably reinforced 6" slab at grade to transfer this thrust from the rear footings to the shear walls in a manner similar to the action of the floor slab.

The shear wall footings are 2'-6" wide by 2'-0" deep and are designed to carry the total vertical load acting on the shear wall.

The continuous strap footings running from front to rear under the columns are reinforced concrete members 4'-0" wide by 4'-0" deep. These footings carry the axial load of the columns above in addition to the torque from the front wall footing. The footing is designed for the maximum condition of loading, including moment and horizontal thrust during plastic action of the front wall, and maximum roof load on the columns.

Due to the rotation of the structure about the rear footing the front end of the strap footings tend to rotate about the first interior column. The ultimate moments of these strap footings is then utilized to develop shears which help resist the overturning forces.

2. <u>Second and Third Floor Slabs</u>

The second and third floor slabs are one-way reinforced concrete slabs 6" deep which span between faces of the floor girders and between the face of floor girders and the interior face of the shear walls as shown by section C-C, Sheet 10 of Appendix 7.

These slabs are designed for dead load plus a 150 lb. per sq. ft. live load, in accordance with the American Concrete Institute Building Code Requirements for Reinforced Concrete (A.C.I.318-47). The test live load will be simulated by coral-filled sand bags to give an average load of 100 lb. per sq. ft. as explained in section 2.4.7. In addition the floor slabs are designed to transmit the front wall reactions in the plane of the slab to the shear walls. They were analyzed by the method of lattice analogy as explained in section 2.4.5 B-1. This analysis yields strains caused by the horizontal edge loads, and from these strains principal stresses were computed. These were added to the stresses caused by the vertical loads. Steel bars were provided to resist the principal tensile stresses, the concrete being adequate to resist the shear and compressive stresses.

As the building rotates dynamically the vertical acceleration of the front wall, and hence the front edge of the floor slabs is approximately three times the acceleration of gravity. This acceleration drops to zero at the first interior columns which take no part in the rotation. The effective forces on the front edge of the slab due to the acceleration of the dead and live loads become 4 times the gravity weight.

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The floor slabs were also checked for buckling stability under the lateral loads as discussed in section 2.4.5 B-4.

3. Roof Slabs and Girders

The concrete roof slab is framed in a manner similar to the typical floor slabs except at the front and rear wall ends where two-way slab action has been taken into account. The roof slab at the front and rear walls in the vicinity of the shear walls will have a large amount of reinforcement due to the necessity for providing dowels to transfer the horizontal loads to the shear wall. On the roof slab and girders the convention design for dead and live load is replaced by a plastic limit design for dead load plus the dynamic roof loads due to blast pressure. The roof slab is 11 inches in depth and was originally designed for a deflection of L/32 under pressures of 125% of the predicted pressure. Later revisions in the pressure loading and a change in the position of the building reduced the maximum expected deflection to near the elastic range. An analysis of the roof panel at 135% of the revised pressure curves indicates that in the region between the front wall and the first line of interior columns some plastic action can be expected. The roof girders are 2'-6" wide by 2'-10" deep, and were also originally designed for a deflection of L/32 at 125% of the predicted pressure curve. Later revisions in the predicted roof pressures have greatly reduced the expected deflections, even under loading of 135% of the latest theoretical pressure curve.

The analysis of a roof system where plastic deformation takes place in the slab and the beams remain essentially elastic may be simplified by considering the slab as supported on unyielding supports, and the beam may be assumed to carry the slab reaction.

If both the slab and its supporting beams are in the elastic range, or if both are in the plastic range, the interaction of the two elements must be considered for an accurate prediction of accelerations, velocities and deflections.

Elastically the problem involves the vibration of a continuous plate on elastic supports having a longer period of vibration than the plate itself. If the ratio of natural frequencies is large the problem can be resolved into two independent analyses of the two components.

Plastically the problem cannot be solved by considering the actions of the two components as independent, because the yielding of the support may be simultaneous and of comparable magnitude and velocity with the plate deflection.

A step-by-step method of solution with very small time increments was established. The method involved the attainment of the

dynamic equilibrium condition for each time interval by means of successive approximations. The problem became more complex for the particular case of the roof system since the deflection of the girders varied from zero at the columns to a maximum at mid-span.

After working through a few typical solutions, using various assumptions as to stiffness, stress-strain time relations, etc., it was found that although there may be large errors in the intermediate steps, a design based on independent action of the slabs and girders would yield a roof system which would be satisfactory for test purposes.

It is felt that the test results will give sufficient data to evaluate with a much higher degree of accuracy, those uncertain factors which minimized the value of the more accurate solution described above. In the final design of the roof slabs and girders, the methods of section 2.5 were used directly.

4. Front and Rear Walls

The front and rear faces of Building 4 are covered by solid monolithic concrete walls. These walls are indicated and detailed on Drawing 60-09-06, Sheets, 1, 10 and 11 of Appendix 7.

To provide adequate room for the two layers of reinforcing bars which are required in each face of each wall and because of the beneficial effects of high mass values, the front and rear walls are one foot thick. Special web reinforcement is provided at all supports, since the shearing stresses exceed the stresses appropriate for plain concrete when the wall develops its full plastic resistance. This condition is typical for one-way slabs subjected to loads comparable to the front wall pressures unless the thickness of the slab is much greater than that required to develop the necessary resisting moments, with reasonable steel percentages. The arrangement of this web reinforcement is as shown on Drawing 60-09-06, Sheets 11 and 12 of Appendix 7. The design of reinforcement is as outlined in Appendix A5-11.

The front walls of Building No. 4 were originally designed for a deflection of L/32 in each story at 125% of the predicted pressure curves. An analysis of the wall panels on the basis of the revised pressure curves indicates these deflections will now be reached at 135% of the predicted blast except for the third story front wall panel which will reach its maximum allowable deflection at a slightly lower percentage on this floor level due to the increase in the duration of the blast impulse. Both front and rear walls are designed as one-way slabs except for the half-panels adjacent to the shear walls which were necessarily designed for two-way action. This two-way action carries part of the horizontal blast load directly to the shear walls, thus reducing the stress concentration which might normally be expected at the front edge of the floor slabs adjacent to the shear walls.

In addition to local blast loads the front wall was designed as a simple beam spanning vertically between the shear walls during dynamic rotation of the building. The loads during this time consisted of the effective forces due to the dead load of the wall and footing, the reflected wall pressure on the projection of the footing, and the shears from the roofs, floors, and strap footings.

The rear wall is of the same thickness and has substantially the same reinforcement as the front wall. It was so designed because the direction of the blast may be unknown on actual installations and to obtain,to a degree of symmetry,approximating actual designs. Because the blast pressure is less on the rear face, the over-reinforced rear wall panels will carry the local blast pressure in or near the elastic range.

The rear wall is also designed as a vertical beam spanning between the shear walls. The wall is loaded at the lower edge by the soil pressure which acts on the footing. During the dynamic rotation of the building, as discussed in Section 2.4.5-E, the vertical soil reaction on the footing and the rear wall is equal to the effective weight of all the dead, live and blast loads which correspond to the rotating portions of the building.

5. Shear Walls

The shear walls of Building No. 4 are solid monolithic concrete walls, as detailed on Drawing No. 60-09-06, Sheets, 1, 10 and 12 of Appendix 7. The shear walls are made one foot thick in order to keep the shearing stresses within reasonable limits in addition to reasons similar to those explained in connection with the front and rear walls. The shear walls receive horizontal shear loads through floor and roof slabs and from the front wall panels. The sum of these horizontal loads minus the inertial resistance due to the horizontal component of the rotational acceleration equals the resistance which must be developed at the foundations to prevent sliding. The difference between this value and the maximum frictional resistance which can be developed by the foundation, will accelerate the building horizontally as discussed in section 2.4.5-F. The total horizontal shear at the base of the shear wall is therefore equal to the maximum value of the developed friction as noted above. For buildings with basements or deep foundations the rotations and translations will probably not be important and the required resistance will be a function only of the applied loads.

Since,for the ultimate condition,the plastic deformation of each floor slab results in a redistribution of the stresses based on the assumption of elasticity, and since the wall itself is also designed for plastic action, the shearing stresses along the wall were assumed uniformly distributed over the full length of the wall parallel to the blast. Analyses based on the lattice analogy bore out this assumption. The effective forces due to the dead weights

and the blast forces from the roof and the interior of Building No. 5 were then combined with the shears to compute the principal stresses. Sufficient reinforcing steel was then added to resist the tensile stresses. Although only one interior wall receives lateral blast pressures, the total effect of this load is small and both walls were therefore made the same. Since the value of the shear strength which can be developed by bond between successive pours is uncertain, diagonal dowels were supplied to resist the horizontal forces at each horizontal construction joint.

6. Columns and Floor Girders

The main purpose of the shear wall building is to obtain test data on the action of shear walls under actual blast loads. The columns and floor girders will not be subjected to large primary deformations except in the front exterior bay, where the rise of the front wall due to the rotation of the rigid shell causes distortions of the floor and roof girders.

Since the major loads are axial, square columns with spiral cores were adopted as most efficient for this condition and as possessing the greatest toughness and shock resistance. The critical design condition for the interior columns is maximum axial load with no moment, while the exterior columns are designed to withstand axial load and moments due to unbalanced floor loads. The roof girder rotations cause local bending failures in the tops of the columns which do not affect the overall axial load capacity.

The columns are designed for their ultimate loads by the methods of plastic theory.

The floor girders are 1'-1" wide by $1'\text{-}11\frac{1}{2}''$ deep and, except for the front exterior bay, are designed to remain elastic during the entire blast cycle.

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A2.3.4 End Cell Buildings - Buildings No. 1 and 7

A. Introduction

The end buildings serve primarily as end seals to prevent the entrance of blast pressures into the interior of Buildings No. 2, 3, 5 and 6 except through the openings specifically provided for that purpose. Rigid reinforced concrete cell-type construction was adopted as most practical, based on considerations of strength, economy and usefulness. Additional advantage has been taken of these end structures by utilizing their exterior surfaces to support numerous test panels which are expected to yield valuable information regarding the behavior of several different types of common wall construction under a sufficiently wide range of blast pressures to cover completely the maximum expected variation in blast pressures.

B. General Arrangement

These buildings are three-story structures, 52 feet deep measured in a direction parallel to the blast and $32\frac{1}{2}$ feet wide. Continuous vertical walls and horizontal slabs divide the interior into $9\frac{1}{2} \times 16 \times 11\frac{1}{2}$ foot cells. The spans of the test panels on the front and rear walls are $11\frac{1}{2}$ and $9\frac{1}{2}$ feet in the vertical and horizontal directions respectively. On the side-walls the test panels span $11\frac{1}{2}$ feet in the vertical by 16 feet in the horizontal direction. The foundation consists of a heavy reinforced concrete mat which projects beyond the exterior walls to provide additional stability against overturning.

C. Details of Framing and Design

1. Foundation

The large pressures from the front and side which act on the end cell buildings result in overturning moments about both the rear and the interior edges of the foundation. To provide stability, the width of the end cell buildings was increased from two to three bays in the early stages of the program. In addition it was necessary to provide massive mat foundations which extend 7, 4 and 3 feet beyond the front, rear and exterior side walls respectively. These projections are acted on by vertical blast pressures which also help to counterbalance the overturning moments caused by the horizontal blast loads. As a result of the combined action of the horizontal and vertical blast loads, soil pressures, which are approximately ten times as high as those which might normally be expected by considering only dead loads and conventional live loads, are created in the vicinity of the rear interior corner of the foundation. However, the load will act for only an

extremely short period of time and it is expected that the foundation soils will be fully adequate at the test site. The behavior of soil under dynamic load is discussed in section 2.4.6.

2. Interior Walls

All interior walls are 12 inches in total thickness, and are designed to withstand successfully the maximum blast pressures which may be applied to the faces of the corresponding test panels which they support. Whenever one of the test panels fails, which a certain number are designed to do at any pressures greater than 2/3 of the basic design values, no further structural damage will take place, nor will there be any interference with the action of the adjacent test panels. The strength required to fulfil their function as structural shear walls was determined by methods similar to those employed in designing Building No. 4, although the loads per wall are much smaller for these buildings.

3. Floor and Roof Slabs

The floor slabs in each story are 8" thick between column lines A and B and 6" between column lines B and D. These slabs are monolithic over the entire floor and are also designed to resist the blast load as described above for the interior walls.

The roof slabs are also continuous in two directions and, in addition to their other functions, serve as test panels for the roof pressures.

4. Exterior Panels

All exterior walls consist of test panels constructed by using reinforced concrete, both plain and reinforced brick, corrugated asbestos, and V-beam sheets. The locations of these panels are shown in Appendix 3, "Key to Location of Wall and Roof Panels," and additional data regarding the design and analysis is given in the accompanying table. The test panels have been designed to reach maximum allowable deflections at varying percentages of the theoretical blast loads and should show many stages of plastic deformation for any blast load greater than the minimum expected.

The shearing stresses in the concrete panels vary from about 150 p.s.i. to 400 p.s.i. and since no shear reinforcement has been provided, the post-test analysis should yield

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valuable information on shear strength of flexural members subjected to impulsive loads.

a. Building No. 1

The nine test panels which make up the front wall vary in total thickness from 9 to 12 inches and have different proportions of reinforcing steel, the percentage ranging from 0.27% to 1.04%. Five panels have two-way reinforcement and the remaining four have one-way reinforcement. The reinforcement in three of these one-way slabs spans in the horizontal direction and the remaining panel is reinforced to span in the vertical direction.

The rear wall of Building No. 1 consists of seven reinforced concrete one-way panels varying in thickness from 8 to $10\frac{1}{2}$ inches and two 16 inch brick panels, one of which is reinforced with single 3/8" diameter bars at the front and rear face of each horizontal joint. In the side wall of Building No. 1 there are six 12 inch thick reinforced concrete two-way panels, and two reinforced brick panels and one plain brick panel. The percentage of steel in all of the reinforced concrete panels is 0.27%. The plain brick panel has a total thickness of 12" and is supported by ledges on all four sides and by a vertical WF steel beam at the center. The two reinforced brick panels have total thickness of 8" and 12" and are reinforced with one 3/8" diameter bar at each face of each horizontal joint. Concrete ledges on three sides, a steel WF beam on the fourth side, and a steel WF beam at the center of the horizontal dimension support the panels against blast loads.

b. Building No. 7

The front walls of Building No. 7 consist of three reinforced concrete two-way test panels with 0.35% steel and six panels constructed of commercial V-beam type siding supported on continuous wood blocks at varying spaces ranging from approximately $1\frac{1}{2}$ to $2\frac{1}{2}$ feet. These blocks are in turn anchored to and supported by reinforced concrete panels, which are designed to withstand the maximum expected blast pressure.

Of the nine test panels which make up the rear wall of Building No. 7, three are one-way reinforced concrete, three are constructed of corrugated asbestos siding supported by continuous wood blocks which rest on reinforced concrete panels, and the remainder are covered by V-beam siding supported by vertical 12 WF 19 and 12 WF 22 steel beams. The vertical wide flange members are in turn

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supported at the top by horizontal 12WF 27 beams and on the bottom by a concrete ledge. The reinforced concrete panels are 10 and $10\frac{1}{2}$ inches in total thickness and the main reinforcement spans in the vertical direction. The corrugated asbestos siding is supported by continuous vertical wooden blocks spaced approximately 1 to $1\frac{1}{4}$ feet apart and anchored to and supported by reinforcement concrete panels.

The side wall of Building No. 7 consists of six V-beam panels supported by vertical steel members which vary in spacing from approximately 3 to 4 feet and range in size from 10WF 25 to 14WF 30. These vertical steel members are supported at the top by horizontal 24WF 76 beams. The remaining three panels are covered by corrugated asbestos supported by continuous wood block which vary in spacing from 8 inches to 12 inches, and which are anchored to and supported by the reinforced concrete back-up panels.

D. Details of Construction

Horizontal one-way test panels are separated from adjacent floor or roof slabs by three inch spaces. For one-way vertical spans the same clearance is provided relative to the walls.

A double layer of closely woven glass fabric is used to cover the openings around the test panels in order to seal the blast pressure, from the interior of the building, from building up behind the test panels. Fourteen 5 inch by 5 inch holes were left in each reinforced concrete backing panel to prevent trapped air from building up interior pressures as the light weight covering material deflects.

The V-beam or corrugated asbestos sheets which cover the front of concrete back-up panels are connected to the wood supports by means of $\frac{1}{2}$ inch diameter bolts spaced from 1 to 2 feet apart depending on their location. These anchor bolts are also embedded in the concrete backing panel and serve to resist any rebound action of the siding. Where steel beams serve as supports, anchorage against rebound has also been provided. Concrete panels which are expected to act elastically are provided additional reinforcement to develop resistance to rebound forces which may cause reversal of moments.

For those test panels designed to develop fixed end moments, dowels are provided from the cross walls and from floor or roof slabs. The action of each exterior panel was thus made independent of the behavior of neighboring panels.

Large shear stress will be developed in the various panels of Buildings No. 1 and 7 but no shear reinforcement was provided. It is expected that the test results will yield information concerning the critical shearing strength of concrete under rapid loading.

To permit entrance into any part of the cell buildings after construction, an access way is provided in each story in the interior walls of each cell. These access ways are provided with timber covers designed to withstand the blast pressure if an exterior panel should fail.

Provision has also been made to leave temporary construction openings in various places to aid in removing formwork from those cells which will be completely closed off during the test.

[redacted]

A2.3.5 Personnel Shelter

A. Introduction

The underground test shelter is designed to provide criteria for judging the suitability and economy of various types of personnel shelters. As the design characteristics of such sections are difficult to assess, a number of sections of different strengths are provided for comparative purposes. In addition to the previously mentioned possibilities of error in blast pressures in the design and analysis, and in the strength of materials under rapid load, there are further errors involved in determining the distribution of vertical and lateral soil pressures resulting from the dynamic surcharge.

B. General Arrangement

The test shelter is arranged into two general types. (See Drawing No. 60-09-11, Sheet 1 of 3, Appendix 7.) The main shelter is divided in two by an open passageway extending, in the direction of the blast, down into the shelter and up out the other side. This entranceway is of uniform width and depth except for a wider section at the entrances to the shelter proper.

On one side of the passageway a heavy structural steel door opens into a shelter formed of solid reinforced concrete one foot thick. The interior is 8 feet high and 8 feet wide, except for small haunches at the corners, and 18 feet long. This area is subdivided by plywood partitions into a series of two air locks and has sleeves and opening through the exterior walls as required for the purposes of the Chemical Corps, U. S. A.

On the other side of the passageway a similar door opens into a series of four circular sections each 8 feet long with an inside diameter of 8'-6", except for a corrugated metal section which has an inside diameter of 7 feet. The four circular sections are separated from each other by concrete walls one foot thick. A small covered access opening is provided through each bulkhead.

The shelter is protected by approximately 6 feet of earth cover.

C. Details of Framing and Design

1. Rectangular Section of Reinforced Concrete

The rectangular section, as described under general arrangement, is composed of a series of one-way and two-way reinforced concrete members. The major portion of the length of the shelter spans one way acting as an open end box with

a load on all sides. At the ends, however, the end walls and the adjacent portions of the side, top and bottom slabs, carry the load by two-way action. The roof, or top of the box, was designed assuming that the full dead load, the full load of the earth cover above the shelter, and the blast pressures, are transmitted directly to the top surface. As the structure deforms active and passive pressure will be built up on the sides which will then act to restrain the top corners. It is assumed that this restraint will develop plastic hinges at the corners before critical damage occurs in the top slab. The sides are designed to resist active earth pressure with a surcharge equal to the blast pressure. The sides must also be strong enough to develop the full plastic moment of the top slab. The bottom is designed for a load distribution which assumes downward pressure of the bottom slab acting against an elastic foundation. The reinforcement required by the design computations is relatively light in spite of the small allowable deflections and the high intensity loading.

The intensity of the load on the sides is important in order to determine the thickness and strength required for the top slab. If the thrust of the sidewalls on the ends of the top slab is high, the reinforcing steel is materially reduced. Accurate knowledge of this side thrust will permit major economies in the design of shelters of this type.

There is some question, in the case of personnel shelters, whether the ultimate capacity can be represented by some reasonable plastic deformation, as was the case in the other buildings; or whether the ultimate capacity must be limited to deflections within the elastic range to prevent possible injury from fragments of spalling concrete. Further tests can clarify this consideration

2. Circular Precast Reinforced Concrete Sections

Each of the circular concrete sections in the shelter is 4 feet long, 8 inches in thickness, and has an 8'-6" inside diameter. These sections are precast and two such units placed end to end make up each of the 8 foot sections between bulkheads. The two units are connected together by means of a continuous grouted joint or key extending around the complete circumference. The outer ends are free of the end bulkheads but a two ply membrane water-proofing is provided to cover the joint between the pipe sections and the bulkhead. Inside and outside layers of reinforcement are provided, each layer being placed 1 inch clear of the surface. The amount of reinforcing differs in the three sections.

The original intention was to use high bond reinforcing bars throughout the shelter. Later it seemed that the use of typical smooth bars as wound in the fabrication of conventional precast concrete pipe would be preferable. Whether commercial precast pipe of this type is used or the units are precast at or near the site, the information obtained from the test will be easily converted by conventional design procedures.

[redacted]

Limit designs were used almost exclusively and the results seemed satisfactory from a practical point of view. To accomplish this design the blast pressure loads were first applied to the top surface with various percentages of this vertical load assumed applied horizontally, against the sides, as an active pressure from a surcharge load. For a comparison the same loads were assumed applied at the top and the sides were assumed as deflecting outward against the passive pressure of an elastic foundation.

From these studies it was found that small changes in steel percentages and, consequently, small changes in total cost would cause major changes in behavior. In view of the uncertainties involved, it was decided to make several sections of varied strength whereby, in the post-test analysis, a variety of deflection, acceleration and earth pressure records could be studied in detail in order to develop more exact design criteria.

3. Circular Corrugated Metal Section

The corrugated metal type, which has a materially lower strength than the concrete sections, was added to obtain additional design data on the effect of the large deflections of a flexible pipe on the distribution of the blast and earth pressures.

The corrugated metal section is made up of 10 gage plate with 1-3/4 inch corrugations at a 6 inch pitch. The circumference consists of four separate pieces lapped 4-3/4 inches and connected by two lines of 3/4 inch round bolts.

4. Blast Resistant Doors

The doors leading from the shelter sections to the passageway are 6'-11" by 2'-9" and consist of a 7/16 inch plate stiffened by vertical and horizontal 3 inch and 5 inch I-sections at 1'-4-5/8 inch centers. The door is hung on two Stanley hinges capable of carrying the weight of the door while in the open position. These hinges are so made that they are capable of flexing and thereby allowing the door to transmit the blast pressures directly to the frame. In the closed position the door is held by the frame for pressures toward the shelter and against rebound forces by a self locking stop at the hinge side and three heavy latches on the opposite side.

Details of the door are shown on Drawing 60-09-11, Sheet 3 of 3, Appendix 7.

As the pressures in the passageway and against the door were not given, pressure gages were installed to determine whether side-on, reduced, or increased reflected pressures exist at this point. Because of the lack of detailed information, the door and walls were designed for side-on pressure while the roof and walls of the passageway were designed using 2/3 of the incident pressure as the unbalanced pressure between the inside and outside faces.

A2.3.6 Steel Mill Building

A. Introduction

Single story shed type buildings framed in steel are commonly used as inexpensive commercial and industrial buildings. These buildings are usually designed for relatively light roof and wind loads and use minimum strength coverings and light supporting systems.

Reports on the behavior of these structures revealed a complete failure of this type of covering under the action of heavy blast loads. Though the coverings failed during the initial phases of the blast, sufficient impulse was transmitted to cause extensive damage to the structural frames.

The original scope of this contract called for a single story steel frame building with adequate strength to resist a blast load of somewhat lesser intensity than that used for the multi-story structure. This building was eventually deleted because of its high cost and because part of the information could be obtained from various parts of the multi-story test structure. However, since a more or less complete study and design was made for this building, the results will be presented in some detail.

B. General Arrangement

This single story steel building, covering an area 25' by 45', was framed by bents 15 feet on center spaced parallel to the direction of the blast. Each bent was formed by a truss 6 feet deep, rigidly attached to wide flange columns 20 feet high. The roof and wall framing consisted of corrugated steel sections supported by wide flange purlins and girts which spanned the distance between the steel bents. The trusses were stabilized along the centerline of the building by a longitudinal truss of the same depth as the bent trusses. Motions in the longitudinal direction were to be prevented by bracing in the planes of the lower chords and sidewalls. Unless the building is extremely long, compared to the bent spacing, this roof and wall bracing rather than the bent framing would ordinarily furnish the lateral resistance to the blast loads.

C. Details of Framing and Design

Steel structures exposed to relatively long duration loads must have a relatively high resistance if plastic distortion is to be kept within reasonable limits. Because of the low mass the action of all parts, except possibly the frame, occurs so quickly that little change takes place in the blast intensity during the action. For this reason the blast pressure is essentially constant for light members and the design can be materially simplified by the use of Dynamic Load Factors (DLF) similar to those developed in section 2.5.2 for deformations in the plastic range or by similar

factors as described by Frankland[42] if the deformations are to be kept within the elastic range. The Dynamic Load Factor in both cases is a function of the natural frequency of the member, the pressure-time relationship, and, in the case of plastic action, is also a function of the ductility of the material.

1. Siding and Roofing

The siding and roofing, whether brittle or ductile, may be designed by methods illustrated in section 2.5.3. The strength of the covering determines the girt and purlin spacing which in turn governs the overall economy of the framing. The table of figure A2.3.6-1 shows the girt spacings required by different siding materials available in the commercial market.

Spans req'd. for commercial siding to resist blast pressures (Spans given to nearest 1")					
Peak Pressure p.s.i.	10	20	30	45	60
V-beam, 18 ga.	4'-4"	3'-1"	2'-6"	2'-0"	1'-9
Corrugated Metal, 18 ga.	2'-0"	1'-5"	1'-2"	0'-11"	0'-10"
Corrugated Asbestos	1'-2"	0'-10"	0'-8"	0'-6"	0'-5"

FIG. A2.3.6-1

2. Girts and Purlins

The siding and roofing has so little mass and such high frequency compared to the girts, that the blast forces will be transferred to the girts and purlins with little distortion. These members, depending on location, must resist impact loads of the type shown in figure A2.3.6-2.

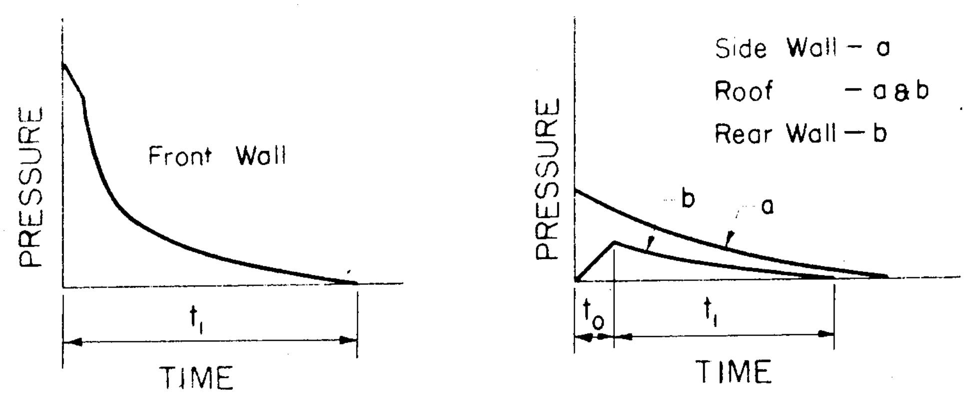


FIG. A2.3.6-2

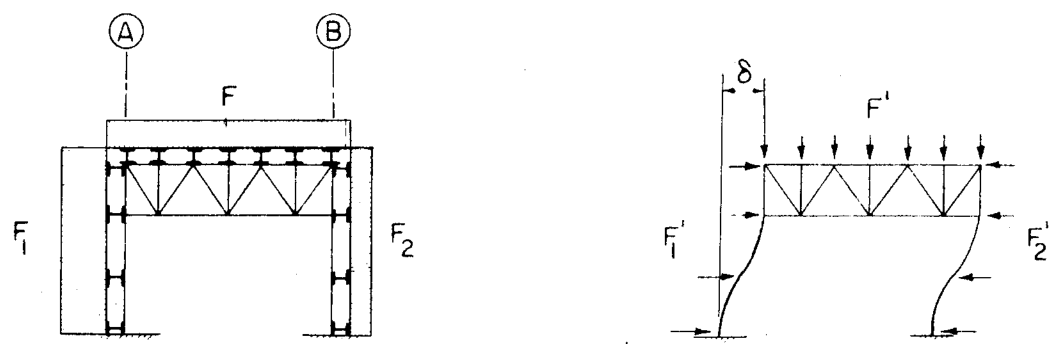
(42) J. M. Frankland, "Effects of Impact on Simple Elastic Structures"

[redacted]

When utilizing plastic range deformation, a preliminary design can readily be made by either the graphical methods discussed in 2.4.5 A8 or by the step by step procedures as described in section 2.5.3. The step-by-step impulse method of analysis indicates that for ductile materials a DLF 1.1 will produce a deflection approaching L/32 under pressure similar in shape to those in figure A2.3.6-2. In most cases the blast pressures will warrant use of wide flange structural shapes for both girts and purlins.

Static tests at Lehigh University (12) indicate local buckling failures do not occur in WF beams stressed well into the plastic range if the loads are applied to the web. The possibility of web buckling may exist, however, if loads are applied to flanges. This possibility may be safely neglected if the duration of the load is short compared to the natural frequency of the beam; but in general it is advisable to provide web stiffeners at points of high shear.

The blast pressure acting on the siding is transferred to the girts as a rapidly oscillating pulse. As shown in figure A2.3.6-3 these members act against the frame and produce reactions equal to F, F_1, and F_2. Theoretically $F \neq F'$, $F_1 \neq F_1'$, and $F_2 \neq F_2'$ until wall and roof vibrations cease, but the oscillation of the covering is so rapid relative to the supporting members that neither the average force nor the work performed on the girts and purlins is sensibly changed if the blast force is used directly as the load on the member as explained in section 2.3.5. For the same reason the girt and purlin reactions on the frame are assumed to be equal to the blast pressure forces after plastic resistance ceases as shown in figure A2.3.6-4. The design is materially simplified by this assumption.



Steel siding & roofing
(a)

(b)

FIG. A2.3.6-3

F, F_1 and F_2 are pressure forces

F', F_1' and F_2' are purlin and girt responses

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(12) W. W. Luxion and B. G. Johnston, "Plastic Behavior of Wide Flange Beams"

[redacted]

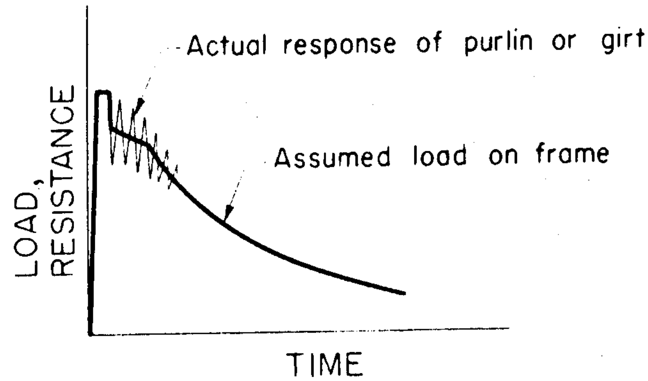


FIG. A2.3.6-4

3. <u>Truss</u>

The major question, in the design of the truss is the desirability of allowing plastic strains in the truss members. While there is no doubt that tension members can take considerable plastic strain, allowance of plastic deformations in compression members seems questionable because of stability requirements. Plastic strain in the tension members will tend to furnish non-rigid supports to the ends of the compression members and limit the forces acting on the compression members.

The forces which the truss must resist are shown in figure A2.3.6-5. If the effective mass of the frame is assumed to be concentrated at the truss level, the chords transmit the net column shear, $V_A - V_B$, which accelerates the mass laterally and develops moments which resist the sidesway. M_A and M_B are the column moments produced by sidesway and roof load. The diagonals and struts perform the same functions as for ordinary loads and in addition act to accelerate the roof mass. The forces, W_A and W_B, are the reactions due to roof loads, F, F' and unbalanced moment $M_A + M_B$.

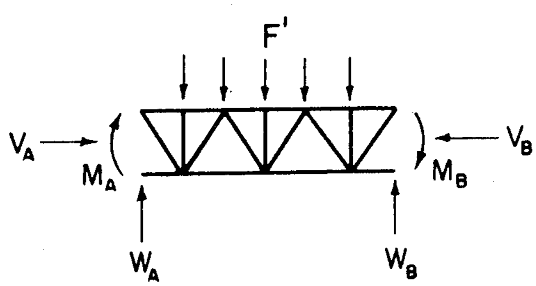


FIG. A2.3.6-5

The rate at which these forces are applied varies:

The column shear, V_A , is a function of forces, F_1 , and frame deflection, δ . The F force has the greatest effect on the column shear and therefore it may be expected that V_A will vary in a similar manner to the front wall blast pressure. As shown in figure A2.3.6-4 the girt resistances tend to slow down the rate of application to the frame of front wall pressures but for practical purposes this action may be neglected.

The column shear, V_B , depends on forces F_2' , and deflection, δ . Both the rear pressure forces, F_2' , and column resistances due to lateral deflection of the frame, δ , take an appreciable time to build up to maximum values. The expected variation of V_B is similar to curve b , figure A2.3.6-2.

The top column moments, M , are dependent on the frame deflection, δ , and forces F' , and either F_1' or F_2' . For the columns along line A the front pressure is predominant if the wall is framed as shown in figure A2.3.6-3. In this case the moments produced at the top of the column by the roof load and sidesway will tend to reverse the moment produced by the front pressure. The variation of M_A may be expected to be similar to the column shear, V_A , (refer to figure A2.3.6-4) except that the roof load and sidesway moments produce a reversal sometime after the frame deflection begins. On the other hand, the moment produced at the top of columns along line B will increase at a relatively slow rate and will not change its sense.

The column reactions, W , are produced by the roof load, F' , and the column moments M_A & M_B : the roof load F' , will have the greatest effect on the vertical shear.

A method for designing trusses to resist the forces described is discussed in section 2.4.5-G. These truss members and truss connections become relatively heavy even for small spans at appreciable distances from the blast center. In figure A2.3.6-6 is shown a portion of the steel building framing required to withstand a relatively light roof pressure of approximately 7 p.s.i.

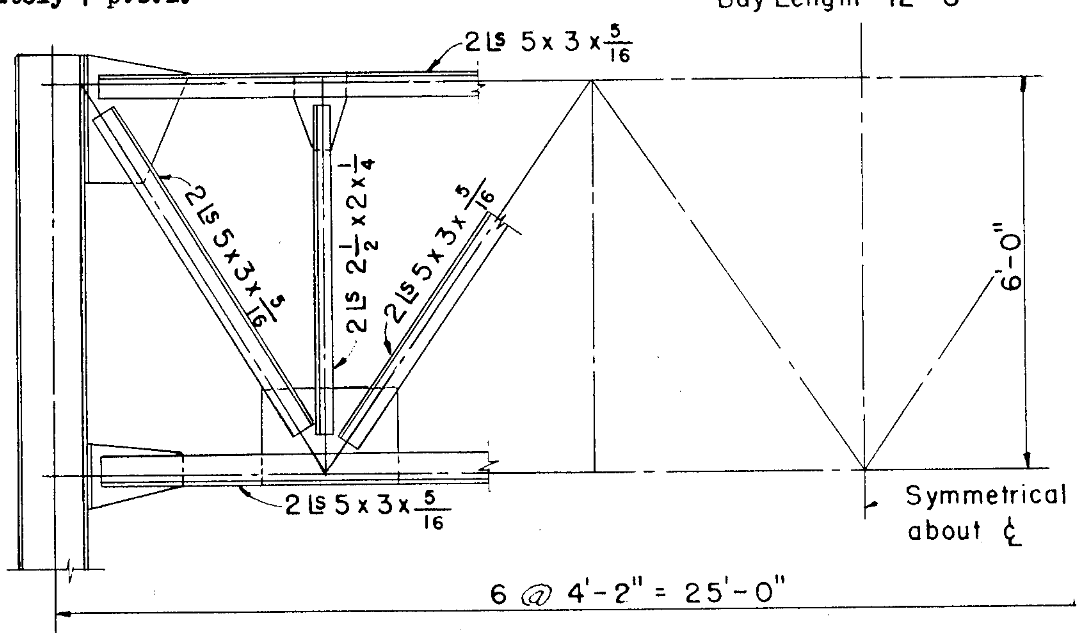


FIG. A2.3.6-6

[redacted]

4. Columns

As may be seen in figure A2.3.6-3b, girt framing details affect the column resistance. The maximum spacing of the girts will be, in general, less than 5' on center, necessitating a framing detail as shown in figure A2.3.6-3a. In this case the front columns (line A) will offer no real resistance to sidesway until the deflection becomes quite large. The concrete wall detail shown in figure A2.3.6-7 may prove superior to the light covering on closely spaced girts in this respect. Note, however, that the girts in this framing must be appreciably larger than for steel siding. Special siding, such as precast panels spanning from the base to the roof level, may be used to provide the framing shown in figure A2.3.6-7. Such precast panels would have the additional advantage of increasing the effective mass of the deflecting structure.

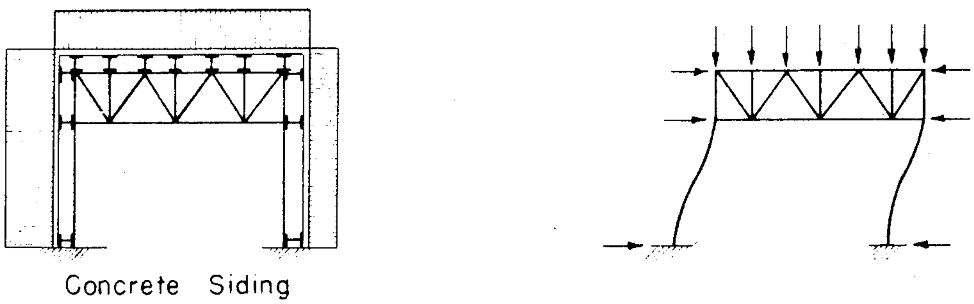


FIG. A2.3.6 -7

The analysis of the frame is similar regardless of the girt framing except for the consideration of the local bending effect on the total resistance against sidesway.

5. Footings

Because the columns required to keep deflection within practical limits are much larger than needed for standard loads, advantage should be taken of the increased resistance provided by fixing the column bases. Since additional weight is needed in the footings to prevent overturning and sliding, the footings may be constructed as continuous strips from the front to the rear, and the bottom ends of the columns can be fixed to these members. Comparative estimates of various frame and footing combinations showed considerable economy in this procedure.

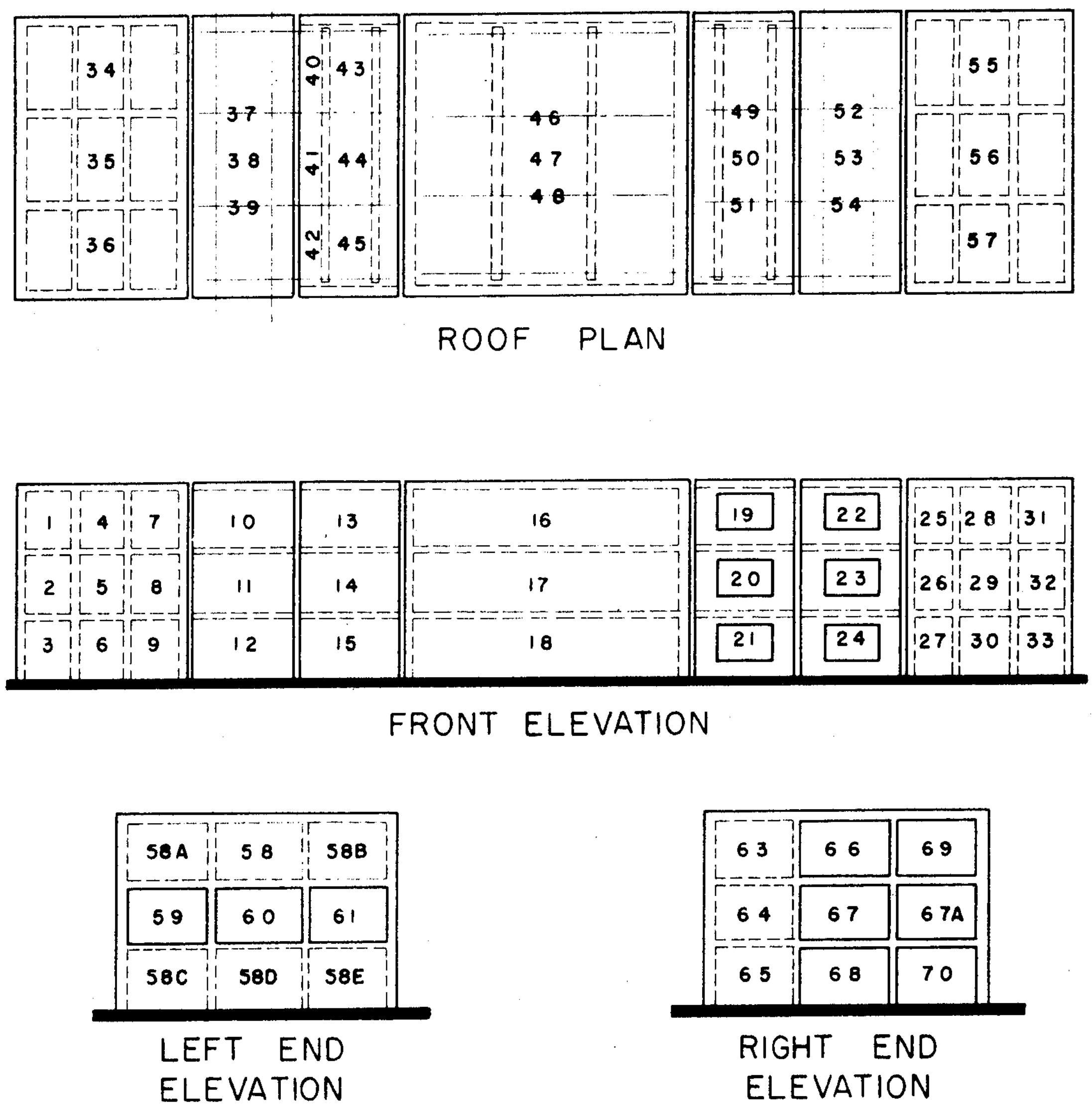
[redacted]

A P P E N D I X 3

SUMMARY OF DESIGN DATA AND RESULTS OF ANALYSIS FOR WALLS, ROOF PANELS AND FRAMES

CONTENTS

A3.1 DESIGN DATA AND RESULTS OF ANALYSIS FOR WALLS AND ROOF PANELS



KEY TO LOCATION OF WALL AND ROOF PANELS

Primes (') in table differentiate rear wall panels from corresponding front wall panels.

Building No.	Panel No.	Material Exposed to Blast	Framing	Thickness	% Reinforcement	Spacing of Supports or Clear Span in Feet	Pressure Curve	Design % of Pressure Curve	Design						Deflection	
									Static			Dynamic				
									f'_c k.s.i	f_s k.s.i	W_R Plastic kips	Dynamic Stress Factor	v p.s.i	Type of Stress Function	δmax. feet	$\frac{\delta}{L}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
1	1	Concrete	✛	12"	0.27	9.5x11.5	A1-11	150%	3	50	175	135%	186	Block	0.251	1/46
	2	"	↔	12"	1.04	11.3	A1-10	150%	3	50	69.4	100%	425	"	0.044	1/258
	3	"	✛	11"	0.39	9.5x11.5	A1-9	150%	3	50	131	135%	220	"	0.525	1/22
								135%							0.251	1/46
	4	"	↔	12"	0.42	9.5	A1-11	125%	3	50	39.2	135%	240	"	0.552	1/17
								100%							0.151	1/63
	5	"	↔	10"	0.52	9.5	A1-10	75%	3	50	31.7	135%	240	"	0.220	1/43
	6	"	↔	12"	0.46	9.5	A1-9	100%	3	50	41.8	135%	256	"	0.300	1/32
	7	"	✛	12"	0.27	9.5x11.5	A1-11	150%	3	50	175	135%	186	"	0.251	1/46
	8	"	✛	9"	0.49	9.5x11.5	A1-10	100%	3	50	129	135%	193	"	0.299	1/38
	9	"	✛	12"	0.27	9.5x11.5	A1-9	150%	3	50	206	135%	218	"	0.274	1/70
								135%							0.132	1/86
								125%							0.076	1/152
	58	"	✛	12"	0.42	11.5x16.0	A1-2	135%	3	50	31	135%	190	Slope	0.011	1/960
								150%							0.017	1/680
					0.27											
	59	Brick	○	8"	0.46	7.5	A1-2	135%	2.5	50	20.7	135%	190	Block	0.108	1/70
								125%							0.063	1/120
								150%							0.272	1/27
	60	"	●	12"	—	7.5	A1-2	—	—	—	—	—	—	—	—	—
	61	"	○	12"	0.28	7.5	A1-2	150%	2.5	50	25.6	135%	150	Block	0.029	1/258
								160%							0.046	1/162
								135%							0.018	1/410
	1'	Concrete	↔	10"		9.5	A1-33									
	2'	"	↔	$10\frac{1}{2}$"		11.5	A1-32									
	3'	"	↔	$10\frac{1}{2}$"		9.5	A1-31									
	4'	"	↔	10"		9.5	A1-33									

[redacted]

Building No.	Panel No.	Material Exposed to Blast	Framing	Thickness	% Reinforcement	Spacing of Supports or Clear Span in Feet	Pressure Curve	Design % of Pressure Curve	Design						Deflection	
									Static			Dynamic				
									f'_c k.s.i	f_s k.s.i	W_R Plastic kips	Dynamic Stress Factor	v p.s.i.	Type of Stress Function	$\delta max.$ feet	$\frac{\delta}{L}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
1	5'	Brick	○	16"		8.5	A1-32									
	6'	"	●	16"	---	8.5	A1-31									
	7'	Concrete	↔	10"		9.5	A1-33									
	8'	"	↔	8"		9.5	A1-32									
	9'	"	↔	8½"		9.5	A1-31									
	34	"	✛	6"	0.63	9.5x16.0	A1-35									
			↔		1.53		A1-35	135%	3	50	25.9	135%	370	Slope	0.013	1/730
	35	"	✛	6"	0.63	9.5x16.0	A1-36									
			↔		1.53			135%	3	50	24.8	135%	354	"	0.035	1/272
	36	"	✛	6"	0.63	9.5x16.0	A1-34									
			↔		1.53			135%	3	50	24.8	135%	354	"	0.202	1/47
2	10	V-Beam	↔	18ga	—		A1-8									
	11	"	↔	18ga	—		A1-7									
	12	"	↔	18ga	—		A1-6									
	10'	"	↔	18ga	—		A1-30									
	11'	"	↔	18ga	—		A1-28									
	12'	"	↔	18ga	—		A1-26									
	37	Concrete	↔	8"												
	38	"	↔	8"			A1-36									
	39	"	↔	8"												
3	13	"	↔	12"	0.64	11.0	A1-8	135%	3	50	57.4	135%	339	Block	0.281	1/39
	14	"	↔	12"	0.88	11.3	A1-7	135%	3	50	68	135%	416	"	0.325	1/35
	15	"	↔	12"	1.16	11.4	A1-6	135%	3	50	79.4	135%	486	"	0.164	1/70
	13'	"	↔	12"	0.64	11.0	A1-30	135%	—	—	—	—	—	—	0.0009	—
	14'	"	↔	12"	0.88	11.3	A1-28	135%	—	—	—	—	—	—	0.0011	—
	15'	"	↔	12"	1.16	11.4	A1-26	135%	—	—	—	—	—	—	0.0014	—

[redacted]

ilding No.	Panel No.	Material Exposed to Blast	Framing	Thickness	% Reinforcement	Spacing of Supports or Clear Span in Feet	Pressure Curve	Design % of Pressure Curve	Design: Static f'_c k.s.i	Design: Static f_s k.s.i	Design: Static W_R Plastic kips	Design: Dynamic Dynamic Stress Factor	Design: Dynamic v p.s.i.	Design: Dynamic Type of Stress Function	Deflection δmax. feet	Deflection $\frac{\delta}{L}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
3	40	Concrete	↔	9"	0.49	3.3	A1-35	135%	3	50	8.6	135%	148	Slope	0.0021	1/1550
	41	"	↔	9"	0.49	3.3	A1-36	135%	3	50	8.6	135%	148	"	0.0027	1/1200
	42	"	↔	9"	0.49	3.3	A1-34	135%	3	50	8.6	135%	148	"	0.064	1/51
	43	"	↔	9"	0.49	8.5	A1-35	135%	3	50	22.2	135%	191	"	0.0025	1/3400
	44	"	↔	9"	0.49	8.5	A1-36	135%	3	50	22.2	135%	191	"	0.0023	1/3700
	45	"	↔	9"	0.49	8.5	A1-34	135%	3	50	22.2	135%	191	"	0.030	1/284
4	16	"	↔	12"	0.64	11.0	A1-5	135%	3	50	52.8	135%	323	Block	0.434	1/25
								125%							0.269	1/41
	17	"	↔	12"	0.88	11.3	A1-4	135%	3	50	68	135%	416	"	0.325	1/35
								125%							0.200	1/57
	18	"	↔	12"	1.16	11.4	A1-3	135%	3	50	71.6	135%	438	"	0.306	1/37
								125%							0.180	1/63
	16'	"	↔	12"	0.64	11.0	A1-29	135%	—	—	—	—	—	—	0.001	—
	17'	"	↔	12"	0.88	11.3	A1-27	135%	—	—	—	—	—	—	0.0011	—
	18'	"	↔	12"	1.16	11.4	A1-25	135%	—	—	—	—	—	—	0.0012	—
	46	"	↔	11"		15.2										
	47	"	↔	11"	0.83	15.2	A1-36	135%	3	50	36.5	135%	247	Slope	0.139	1/109
	48	"	↔	11"		15.2										
5	19	"	↔	12"	0.89	11.0	A1-42	135%	3	50	51.3	135%	314	Block	0.222	1/50
	20	"	↔	12"	0.82	11.4	A1-41	135%	3	50	57.5	135%	339	"	0.212	1/54
	21	"	↔	12"	0.95	11.5	A1-40	135%	3	50	60	135%	367	"	0.353	1/33
	19'	"	↔	12"		11.0	A1-45									
	20'	"	↔	12"		11.4	A1-44									
	21'	"	↔	12"		11.5	A1-43									
	49	"	↔	9"												
	50	"	↔	9"			A1-48									
	51	"	↔	9"												

[redacted]

Building No.	Panel No.	Material Exposed to Blast	Framing	Thickness	% Reinforcement	Spacing of Supports or Clear Span in Feet	Pressure Curve	Design % of Pressure Curve	Design – Static – f'_c k.s.i	Design – Static – f_s k.s.i	Design – Static – W_R Plastic kips	Design – Dynamic – Dynamic Stress Factor	Design – Dynamic – v p.s.i.	Design – Dynamic – Type of Stress Function	Deflection – $\delta max.$ feet	Deflection – $\frac{\delta}{L}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
6	22	V-Beam	↔	18ga	—		A1-42									
	23	"	↔	18ga	—		A1-41									
	24	"	↔	18ga	—		A1-40									
	22'	"	↔	18ga	—		A1-45									
	23'	"	↔	18ga	—		A1-44									
	24'	"	↔	18ga	—		A1-43									
	52	Concrete	↔	8"												
	53	"	↔	8"			A1-48									
	54	"	↔	8"												
7	25	V-Beam	↔	18ga	—	2.0	A1-11	80%	—	40	15.2	160%	—	Slope	0.025	1/79
								100%							0.046	1/43
	26	"	↔	18ga	—	2.4	A1-10	60%	—	40	12.8	160%	—	"	0.027	1/89
	27	"	↔	18ga	—	1.5	A1-9	150%	—	40	20.9	160%	—	"	0.009	1/159
	28	"	↔	18ga	—	1.9	A1-11	100%	—	40	16.3	160%	—	"	0.025	1/74
								120%							0.068	1/28
	29	"	↔	18ga	—	1.7	A1-10	120%	—	40	17.9	160%	—	"	0.016	1/107
								150%							0.094	1/18
	30	"	↔	18ga	—	2.5	A1-9	50%	—	40	12	160%	—	"	0.029	1/92
								60%	—						0.059	1/43
	31	Concrete	↔↕	12"	0.35	9.5x11.5	A1-11	150%	3	50	206	135%	219	Block	0.119	1/96
	32	"	↔↕	12"	0.35	9.5x11.5	A1-10	150%	3	50	206	135%	219	"	0.237	1/48
	33	"	↔↕	12"	0.35	9.5x11.5	A1-9	150%	3	50	206	135%	219	"	0.269	1/42
	63	Transite	↔	—	—	1.1	A1-2	172%	—	—	2.96	—	—	—	—	—
	64	"	↔	—	—	0.9	A1-2	113%	—	—	2.40	—	—	—	—	—
	65	"	↔	—	—	0.7	A1-2	74%	—	—	3.86	—	—	—	—	—
	66	V-Beam	↔	18ga	—	3.2	A1-2	80%	—	40	9.6	160%	—	Slope	0.034	1/94
								100%							0.060	1/53
								120%							0.203	1/16

[redacted]

Building No.	Panel No.	Material Exposed to Blast	Framing	Thickness	% Reinforcement	Spacing of Supports or Clear Span in Feet	Pressure Curve	Design % of Pressure Curve	Design: Static: f'_c k.s.i	Design: Static: f_s k.s.i	Design: Static: W_R Plastic kips	Design: Dynamic: Dynamic Stress Factor	Design: Dynamic: v p.s.i.	Design: Dynamic: Type of Stress Function	Deflection: δ max. feet	Deflection: $\frac{\delta}{L}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
7	66	Girt	↕	12WF	27 —	8.5	A1-2	65%	—	40	—	160%	—		0.026	1/326
								100%							0.040	1/212
								125%							0.050	1/170
	67	V-Beam	↔	18ga	—	2.8	A1-2	125%	—	40	10.8	160%	—	Slope	0.045	1/63
								135%							0.063	1/44
								150%							0.159	1/18
	67	Girt	↕	12WF	27 —	8.5	A1-2	60%	—	40	—	160%	—		0.014	1/605
								100%							0.023	1/368
								135%							0.032	1/265
	68	V-Beam	↔	18ga	—	3.8	A1-2	75%	—	40	8.2	160%	—	Slope	0.077	1/49
								80%							0.145	1/26
	68	Girt	↕	10WF	25 —	8.5	A1-2	60%	—	40	—	160%	—		0.028	1/308
								80%							0.037	1/228
								100%							0.046	1/183
	69	V-Beam	↔	18ga	—	2.5	A1-2	135%	—	40	12	160%	—	Slope	0.025	1/101
								150%							0.034	1/75
	69	Girt	↕	14WF	30 —	8.5	A1-2	60%	—	40	—	160%	—		0.009	1/942
								100%							0.015	1/565
								150%							0.023	1/368
	70	V-Beam	↔	18ga	—	2.3	A1-2	140%	—	40	13	160%	—	"	0.017	1/140
	25'	"	↔	18ga	—	4.8	A1-33									
	26'	"	↔	18ga	—	4.3	A1-32									
	27'	"	↔	18ga	—	3.9	A1-31									
	28'	Transite	↔	—	—	—	A1-33									
	29'	"	↔	—	—	—	A1-32									
	30'	"	↔	—	—	—	A1-31									
	31'	Concrete	↔	10"		11.5	A1-33									

[redacted]

Building No.	Panel No.	Material Exposed to Blast	Framing	Thickness	% Reinforcement	Spacing of Supports or Clear Span in Feet	Pressure Curve	Design of Pressure Curve	Design						Deflection	
									Static			Dynamic				
									f'_c k.s.i	f_s k.s.i	W_R Plastic kips	Dynamic Stress Factor	v p.s.i.	Type of Stress Function	δmax. feet	$\frac{\delta}{L}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
7	32'	Concrete	↔	10½"		11.5	A1-32									
	33'	"	↔	10½"		11.5	A1-31									
	55	"	✢	6"	0.63	9.5x16.0	A1-35									
			↔		1.54			135%	3	50	24.8	135%	355	Slope	0.063	1/150
	56	"	✢	6"	0.63	9.5x16.0	A1-36									
			↔		1.53			135%	3	50	24.8	135%	354	"	0.035	1/272
	57	"	✢	6"	0.63	9.5x16.0	A1-34									
			↔		1.54			135%	3	50	24.8	135%	355	"	0.202	1/42

General Notes:

Rear wall panels corresponding to front wall panels have same designation with addition of a prime - e.g.; front wall panel number n corresponds to rear wall panel n'.

All data in table is based on Pressure-Time curves - Issue No. 5 - Appendix I, except as noted.

Legend:

† Data based on Issue No. 6 curves.

* Shear reinforcement provided.

⊕ Panels checked by use of dynamic load factors based on natural frequency of member and blast pressure curves.

↔ Indicates one-way slab or one-way portion of rectangular slab, or one-way framing for V-beam and corrugated asbestos sheets.

↕ Indicates simple span framing for girts.

✢ Indicates two-way square slab or two-way portion of rectangular slab.

o Indicates reinforced brick panel.

● Indicates unreinforced brick panel.

[redacted]

A3.2 Total Displacements of Frames under Various Loadings

Building	Floor	Design % of Pressure Curve		
		65%	100%	135%
2	R	0.401	0.860	1.626
	3	0.207	0.486	1.163
	2	0.072	0.198	0.769
3	R	0.400	1.190	2.799
	3	0.280	0.776	2.194
	2	0.118	0.347	1.180
5	R	0.140	0.392	0.895
	3	0.062	0.320	0.728
	2	0.013	0.194	0.503
6	R	0.215	0.359	0.582
	3	0.119	0.188	0.309
	2	0.043	0.067	0.097

General Notes

Displacements shown are in feet and are based on Issue No. 5 pressure-time curves for Buildings No. 2 and 3 and Issue No. 6 pressure-time curves for Buildings No. 5 and 6.

The design unit stresses for Buildings No. 2 and 6 are 50,000 p.s.i. under the initial blast and 40,000 p.s.i. on rebound. The design unit stresses for Buildings No. 3 and 5 are 50,000 p.s.i. for steel and 3,000 p.s.i. for concrete.

[redacted]

APPENDIX 4

INSTRUMENTATION DRAWINGS

AND

TABLE OF ACCELERATIONS AND DISPLACEMENTS

CONTENTS

[redacted]

TABLE OF ACCELERATIONS & DISPLACEMENTS

BUILDING 1

Location	Acceleration Ft/Sec^2			Deflection Ft.		
Wall Panels:						
% of Blast between col. lines A,B floors 1,2		135% +1060 - 350	150% +1170 - 260		135% 0.01	150% 0.02
% of Blast between col. lines 2,3 floors 1,2		100% +1200 - 620	135%	75%	100% 0.30	135% Failure
% of Blast between col. lines 2,3 floors 2,3	75% +1100 - 490	100%	135%	75% 0.22	100% Failure	135% Failure
% of Blast between col. lines 3,4 floors 1,2		135% +1700 - 850	150% +2100 - 850		135% 0.13	150% 0.27
% of Blast between col. lines 3,4 floors 2,3		100% +2000 - 690	135%		100% 0.30	135% Failure
% of Blast between col. lines 2,3 floors 3, roof					100% 0.16	125% Failure
% of Blast between col. lines 3,4 floors 3, roof			150% +2280 - 960			
Roof Panels:						
% of Blast between col. lines A,B lines 3,4			135% +2070 - 620			135% 0.10
% of Blast between col. lines B,C lines 3,4			135% + 560 - 532			135% 0.04
% of Blast between col. lines C,D lines 3,4			135% + 203 - 229			135% 0.01

[redacted]

BUILDING 2

Location	Acceleration Ft/Sec^2			Deflection Ft.		
% of Blast	65%	100%	135%	65%	100%	135%
Frame						
2nd Floor	+150 -100	+230 - 85	+310 - 80	.07	.21	.77
3rd Floor	+170 -110	+260 - 90	+350 -100	.16	.32	1.17
Roof	+130 -130	+200 -130	+270 -140	.25	.43	1.66

[redacted]

BUILDING 3

Location	Acceleration Ft/Sec^2			Displacement Ft.		
% of Blast	65%	100%	135%	65%	100%	135%
Frame:						
2nd Floor	+ 86 - 35	+ 120 - 40	+ 140 - 40	.12	0.35	1.18
3rd Floor	+102 - 56	+ 130 - 65	+ 180 - 50	.28	0.83	2.20
Roof	+ 67 - 38	+ 110 - 55	+ 155 - 55	.40	1.18	2.80
Wall Panels: (Front)						
1st to 2nd Floors		+ 590 -1020	+1400 - 650		.025	0.16
2nd to 3rd Floors		+ 730 - 855	+1600 - 700		.037	0.33
3rd to Roof		+1150 - 525	+1700 - 620		.140	0.28
Wall Panels: (Rear)						
1st to 2nd Floors					at 1/4 point	0.008
2nd to 3rd Floors					"	0.007
3rd to Roof					"	0.005
Roof Slabs:						
Front Center Span			+ 810 - 250			.030
Front Cantilever			+1240 - 360			.065
Center Center			+ 150 - 175			.003
Center Cantilever			+ 140 - 170			.003
Rear Center			+ 85 - 75			.003
Rear Cantilever			+ 275 - 345			.003
Roof Girders Front			+1155 -1035			.060
Center			+ 765 - 770			.072
Rear			+ 300 - 270			.024

[redacted]

BUILDING 4

Location	Acceleration Ft/Sec^2			Displacement Ft.		
% of Blast	65%	100%	135%	65%	100%	135%
Wall Panels: (Front)						
1st to 2nd Floors			+1600 - 700		at 1/4 point	0.16
2nd to 3rd Floors			+1600 - 700		"	0.17
3rd to Roof			+1800 - 600		"	0.22
Wall Panels: (Rear)						
1st to 2nd Floors						0.14
2nd to 3rd Floors						0.13
3rd to Roof						0.12
Roof Slab at col. line B between col. lines 10,11			+ 735 - 315			.15
at col. line B,C between col. lines 10,11			+ 450 - 230			.07
at col. line C between col. lines 10,11			+ 285 - 235			.03
Roof Girder at col. lines 10,11 between col. lines B,C			+ 520 - 305			.16
Displacement of: Roof relative to 3rd Floor						
Front, Center, Rear						.003
3rd relative to 2nd						
Front, Center, Rear						.01
2nd relative to 1st						
Front, Center, Rear						.02

[redacted]

BUILDING 5

Location	Acceleration Ft/Sec^2			Deflection Ft.		
% of Blast	65%	100%	135%	65%	100%	135%
Frame:						
2nd Floor	+59 -18	+ 75 - 26		.04	.19	
3rd Floor	+74 -33	+ 95 - 45		.06	.32	
Roof	+43 -39	+ 66 - 31		.14	.39	
Wall Panels Front Wall From 1st to 2nd Floor		+865 -515	+1150 - 785		.06	.35
2nd to 3rd		+805 -760	+1475 - 680		.04	.20
3rd to Roof		+940 -685	+1610 - 625		.06	.26

BUILDING 6

Location	Acceleration Ft/Sec^2			Deflection Ft.		
% of Blast	65%	100%	135%	65%	100%	135%
Frame:						
2nd Floor	+ 90 - 52	+140 - 80	+190 -110	.04	.07	.10
3rd Floor	+104 - 75	+160 -108	+215 - 96	.12	.19	.31
Roof	+ 72 -150	+110 -140	+150 -175	.21	.36	.58

[redacted]

BUILDING 7

Location	Acceleration Ft/Sec^2			Deflection Ft.		
<u>Wall Panels:</u>						
% of Blast	60%	80%	100%	60%	80%	100%
between col. lines B,C Floors 1,2	±10650	±14200		0.028	0.037	Failure
% of Blast	60%	100%	135%	60%	100%	135%
between col. lines B,C Floors 2,3	± 7750	±12900	±17400	0.014	0.023	0.032
% of Blast	60%	100%	150%	60%	100%	150%
between col. lines C,D Floors 3rd to Roof	± 6710	±11200	±16800	0.01	0.015	0.023
% of Blast	60%	100%	150%	60%	100%	150%
between col. lines B,C Floors 3rd to Roof	± 9070	±13950	±17450	0.026	0.040	0.050
% of Blast	100%	150%		100%	150%	
between col. lines C,D Floors 1,2	±10700	±16000				

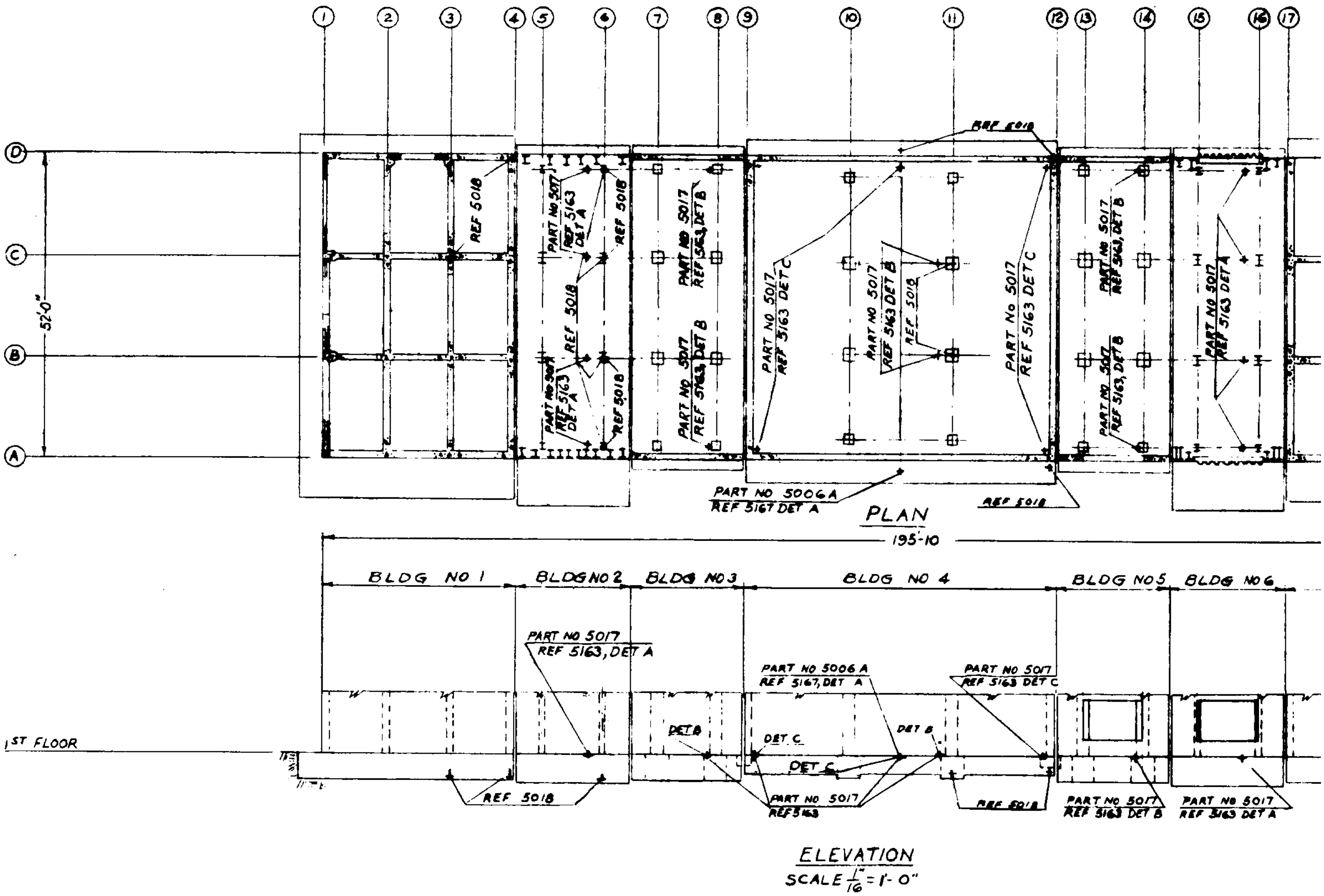
[redacted]

TABLE OF STRAINS IN COLUMNS

BUILDING 2

Location & % Strain		ϵ_T = Tensile Strain ϵ_C = Compression	ϵ_V = Shear
Mid-height of columns from 3rd Floor to Roof			
Flanges (Front & Rear Faces Same)			**Web**
Cols. at	A	ϵ_C = 0.074% ϵ_T = 0.005%	ϵ_V = 0.11%
	B	ϵ_C = 0.084%	ϵ_V = 0.14%
	C	ϵ_C = 0.070%	ϵ_V = 0.14%
	D	ϵ_C = 0.042%	ϵ_V = 0.11%
Columns from 2nd to 3rd Floors			**Web**
at col. line	A		ϵ_V = 0.15%
	B		ϵ_V = 0.21%
	C		ϵ_V = 0.21%
	D		ϵ_V = 0.15%
Columns from 1st to 2nd Floors			**Web**
at col. line	A		ϵ_V = 0.15%
	B		ϵ_V = 0.20%
	C		ϵ_V = 0.20%
	D		ϵ_V = 0.15%
Recommended Range of Instruments			
ϵ_C & ϵ_T	0.2%	(approx. elastic strains)	
ϵ_V	0.3%		

[redacted]



Approved

By

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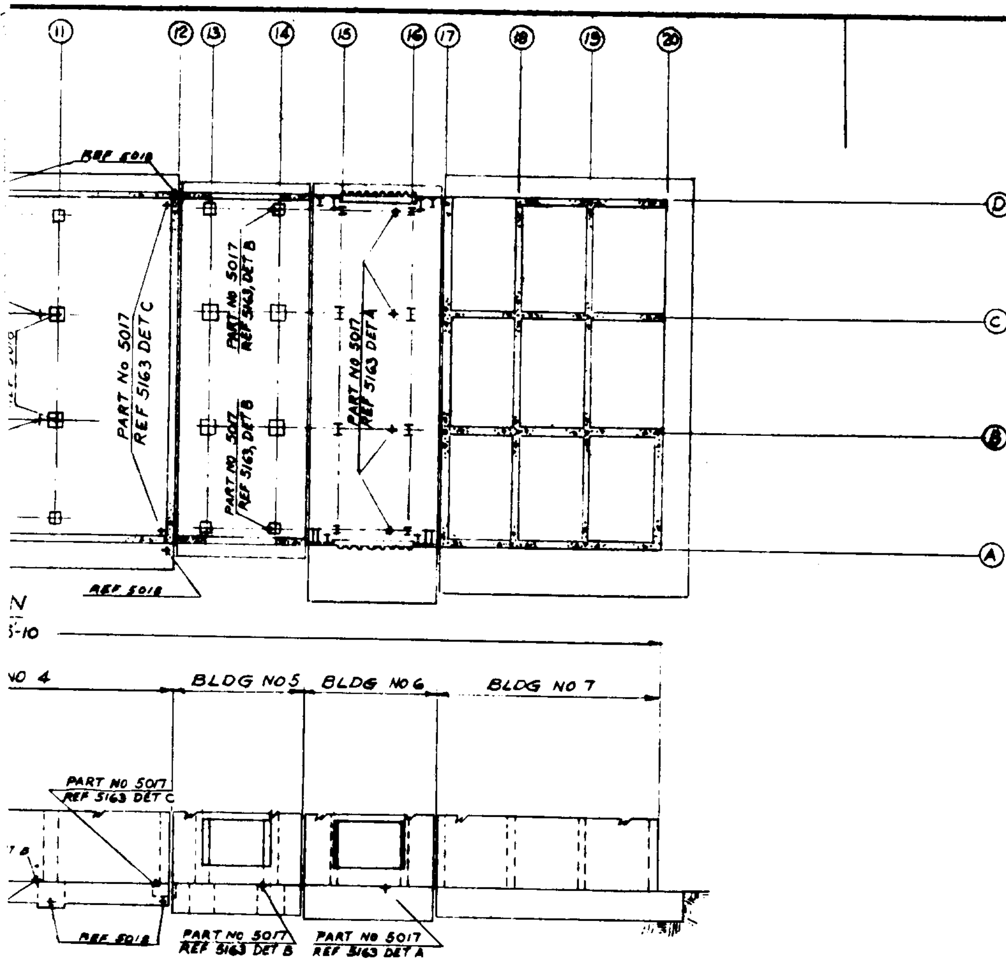


Approved Bruce D Jones
Project 3.1 Officer

By Boyd C Anderson

LOCATION OF INSTRUMENT MOUNTINGS IN FOOTING

HOLMES & NARVER
INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

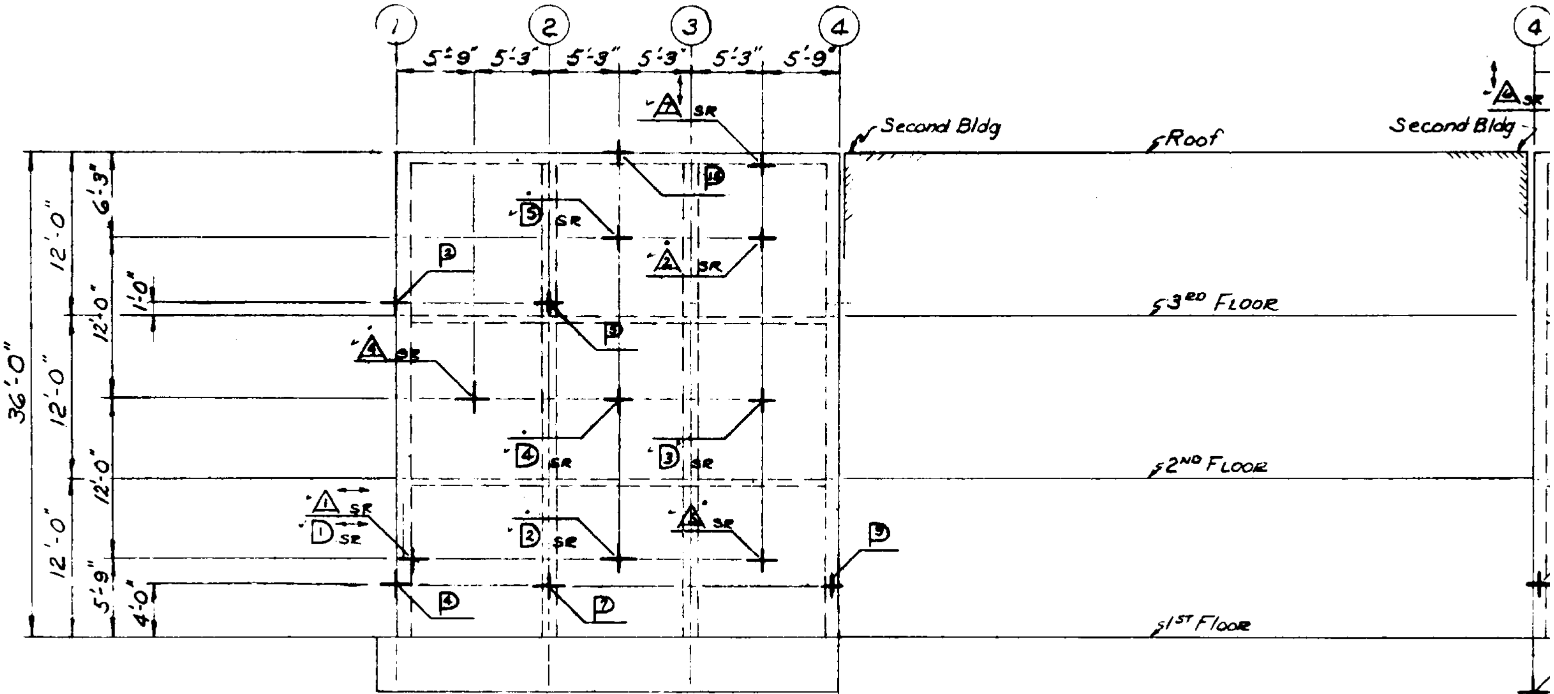
BLDG. 3.1.1

REVISIONS

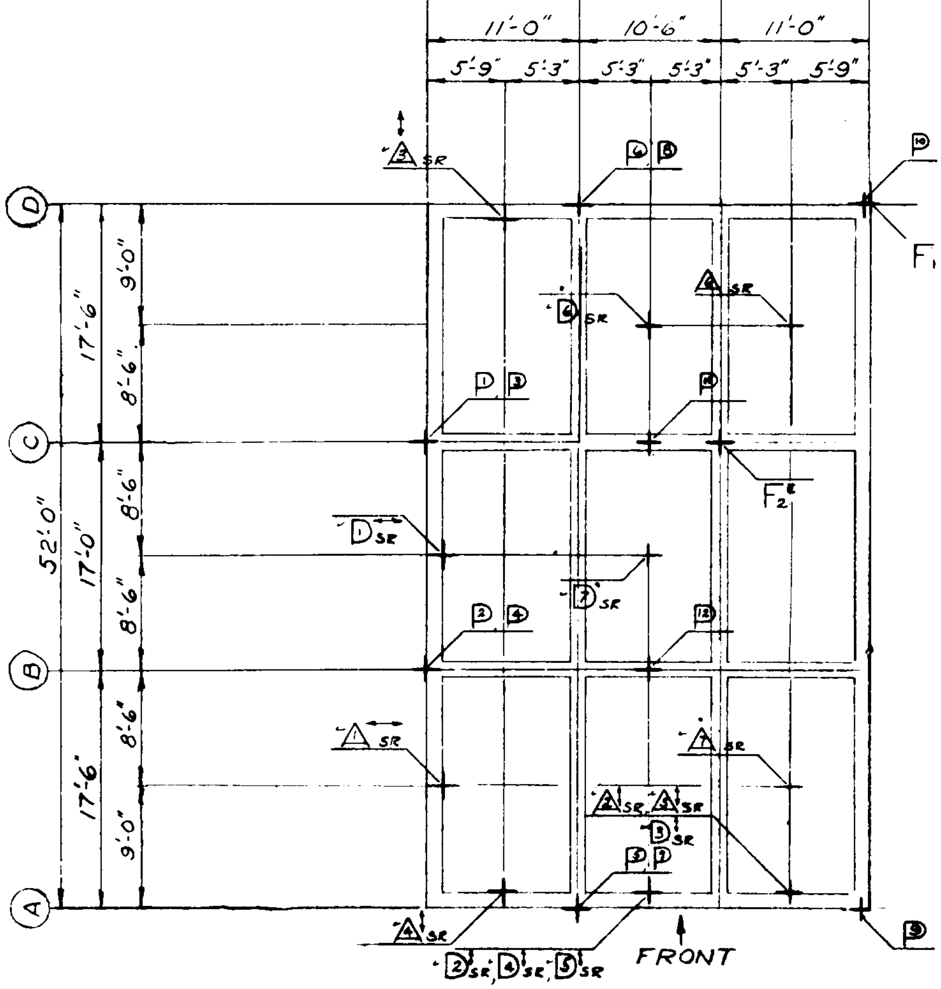
NO.	DATE	DESCRIPTION
1	[illegible]	[illegible]
2	[illegible]	REVISED BLDG #4
3	[illegible]	CORRECTED NOTATIONS
	[illegible]	AS BUILT AS SHOWN

840 4-7-50 5150

[redacted]



FRONT ELEVATION

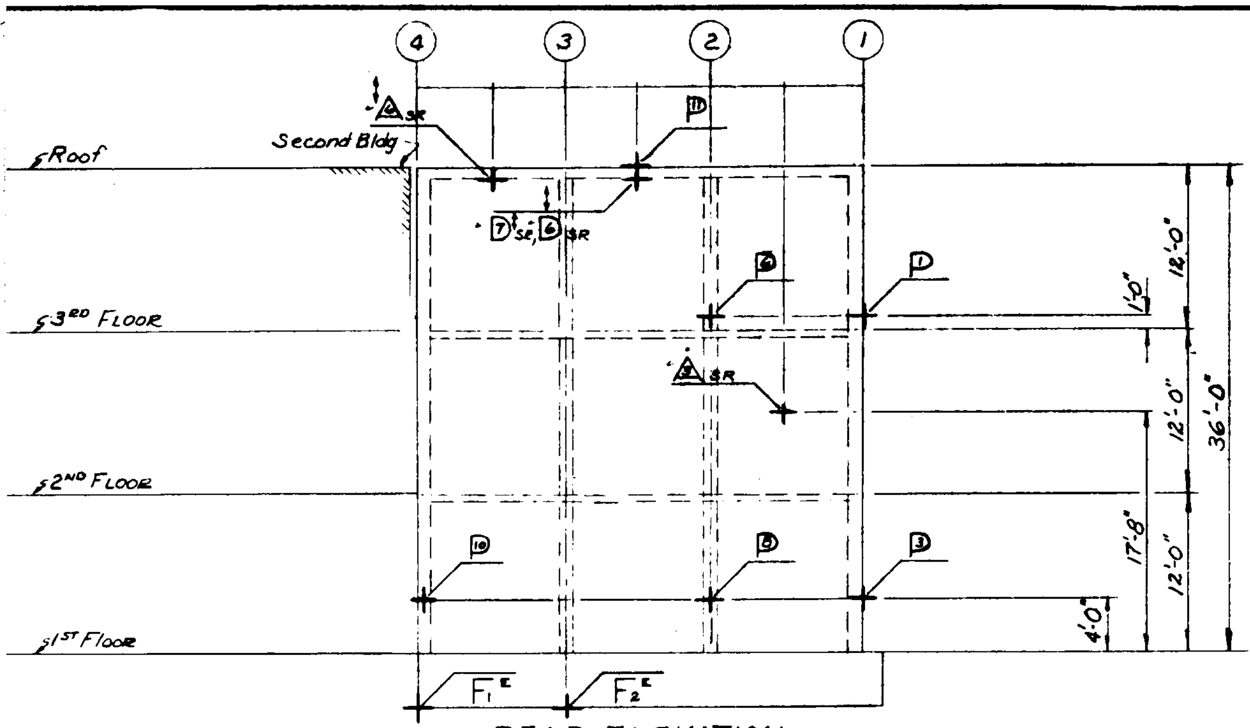


ROOF PLAN

Approved: Bruce ... Projec...

By Boyd ...

[redacted]



REAR ELEVATION

F - 1, 2	—	5018
D - 2, 4, 6, 8, 9, 11	B	5160
D - 1, 3, 5, 7, 10, 12	A	5160
$\triangle_{SR}$ - 6, 7	D	5159
$\triangle_{SR}$ - 1, 2, 3, 4, 5	C	5159
D_{SR} - 6, 7	B	5159
D_{SR} - 1, 2, 3, 4, 5	A	5159
INSTRUMENT	DETAIL	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

REVISIONS		
NO.	DATE	DESCRIPTION
1	5/10/56	Revised A & B details for P
2	3/26/57	CORRECTED NOTATIONS
	6/6/57	AS BUILT AS SHOWN

HOLMES & NARVER
INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

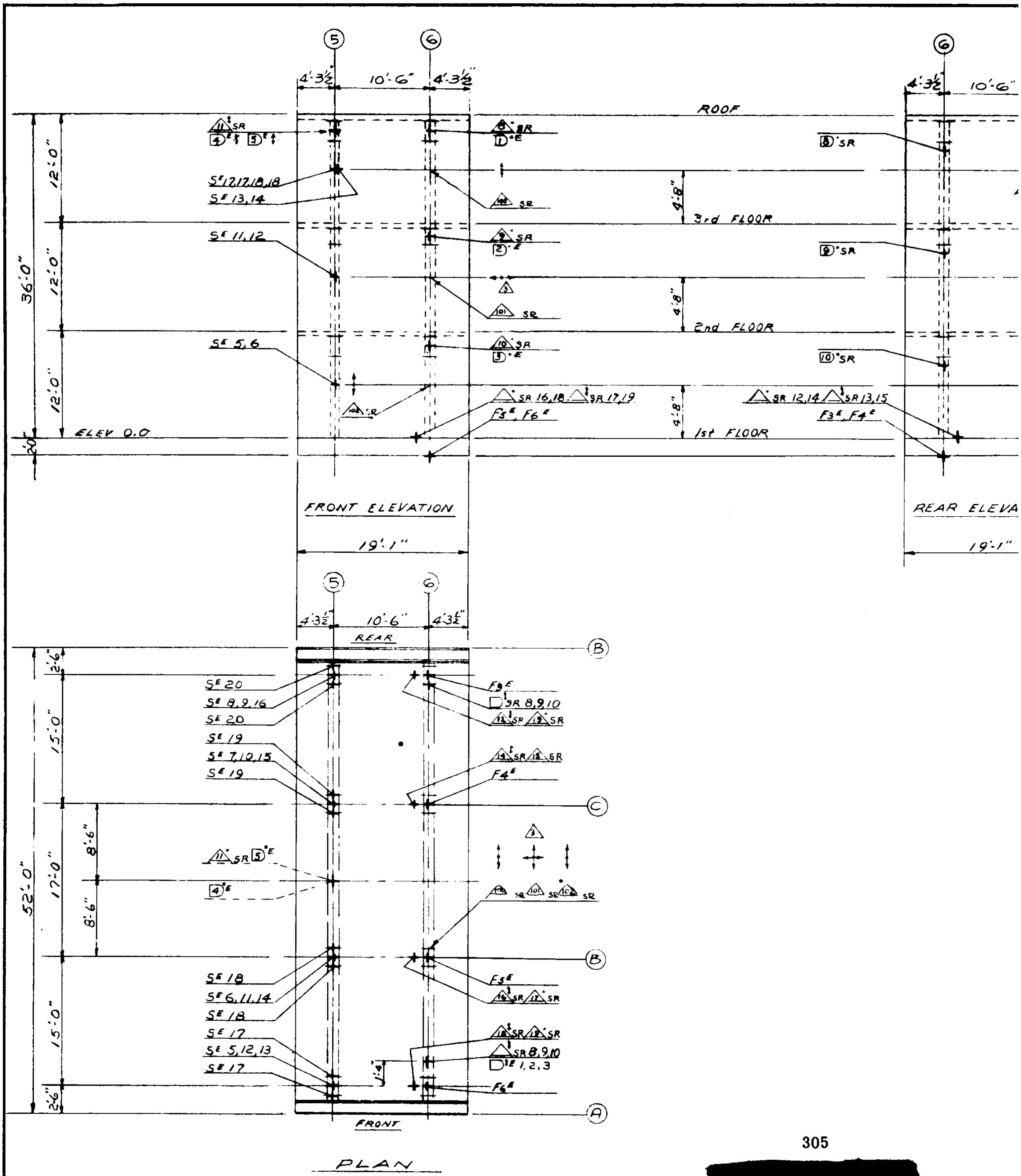
BLDG 3.1.1 (1)

H.E.K. | 3-31-56 | 840F | 5151

Approved: Bruce D Jones
Project 3.1 Officer

By Boyd G Anderson

[redacted]



5
6
4'-3½"
10'-6"
4'-3½"
ROOF
3rd FLOOR
2nd FLOOR
1st FLOOR
ELEV 0.0
36'-0"
12'-0"
12'-0"
12'-0"
2'-0"
4'-8"
4'-8"
4'-8"
SE 12,17,18,18
SE 13,14
SE 11,12
SE 5,6
SR 16,18
SR 17,19
F3E, F6E
SR 12,14
SR 13,15
F3E, F4E
FRONT ELEVATION
REAR ELEVA
19'-1"
19'-1"
REAR
FRONT
B
C
B
A
52'-0"
2'-6"
15'-0"
17'-0"
8'-6"
8'-6"
15'-0"
2'-6"
1'-4"
SE 20
SE 8,9,16
SE 20
SE 19
SE 7,10,15
SE 19
SE 18
SE 6,11,14
SE 18
SE 17
SE 5,12,13
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F5E
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F6E
PLAN

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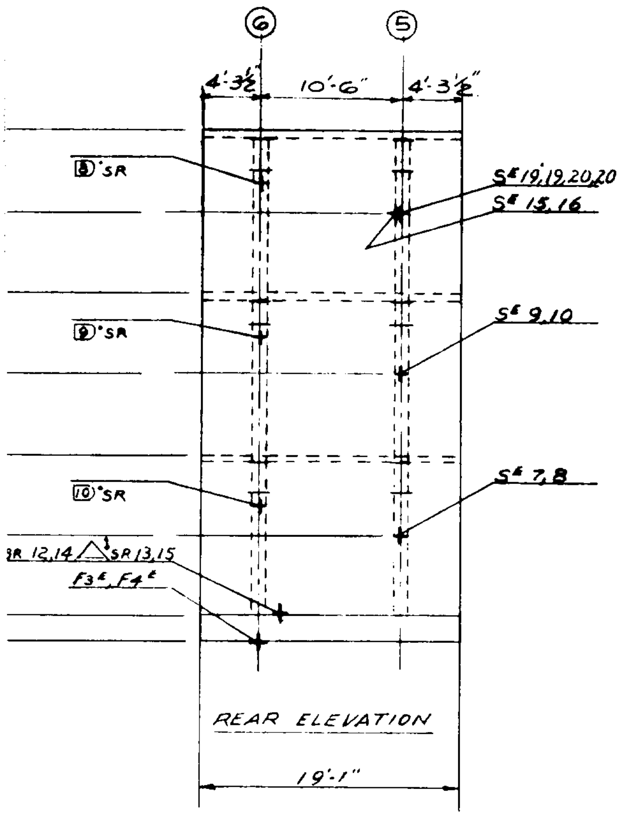

INSTRUMENT	DET	SHEET
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DSR - 8, 9, 10	A	5162
ASR - 8, 9, 10 DE - 1, 2, 3	B	5162
ASR - 11 DE - 4, 5	C	5162
ASR - 12, 13, 14, 15, 16, 17, 18, 19	A	5163
F - 3, 4, 5, 6	–	5018
S - 5 to 20 inclusive	–	–

LOCATION OF INSTRUMENT MOUNTINGS

NO.	DATE	DESCRIPTION
1	4/15/51	Changed notation 3-1 to 16 to 3-5 to 20 incl.
2	5/3/51	Add instruments no. 101, 102 & Delete periods
3	5/8/51	REVISED AS NOTED AS BUILT AS SHOWN

HOLMES & NARVER INCORPORATED ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

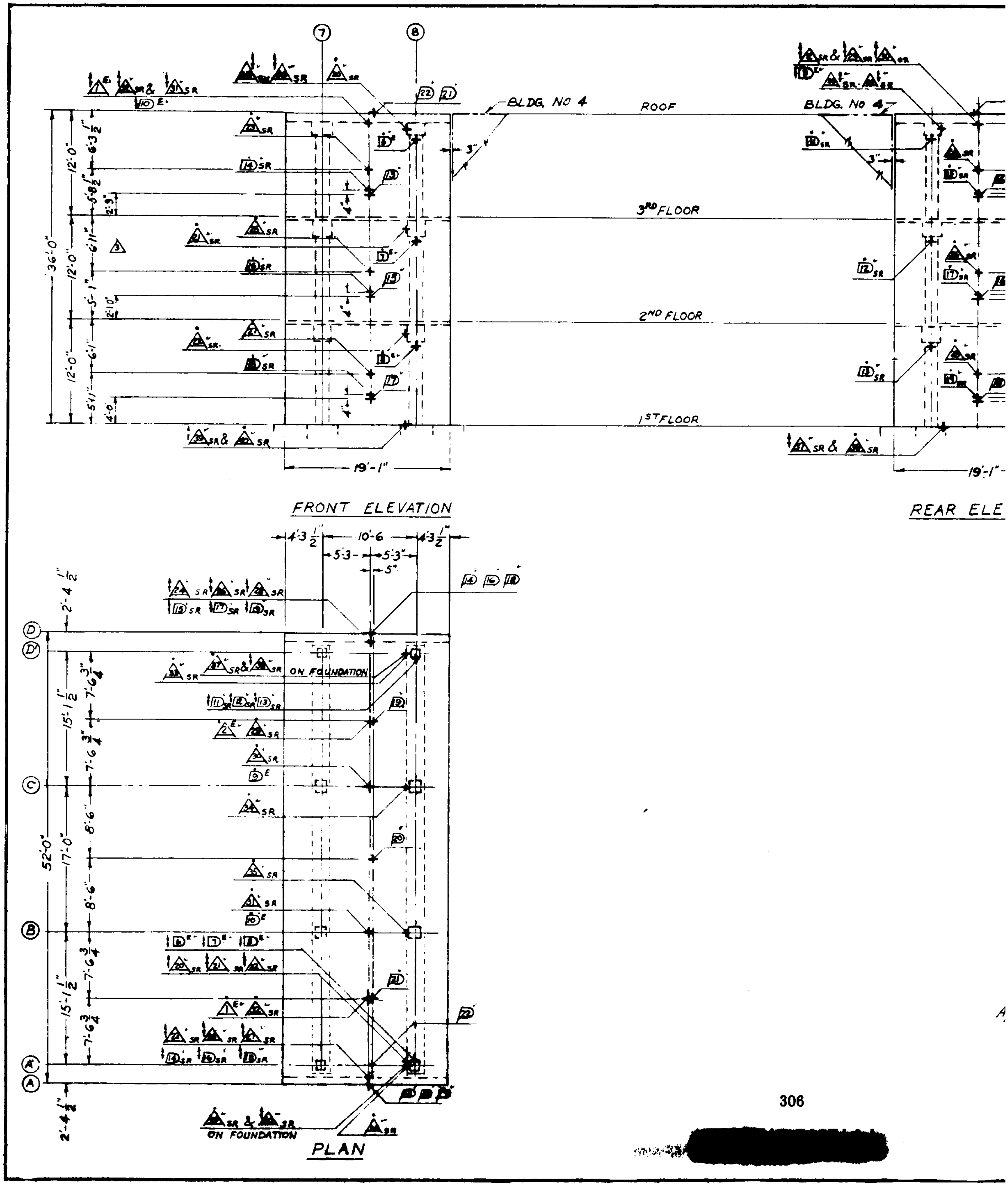
BLDG . 3.1.1 (2)

DR. ROSE | CH. L.V. | DATE 5-31-50 | SHEET NO. 5152
JOB NO. 840F | APPROVED

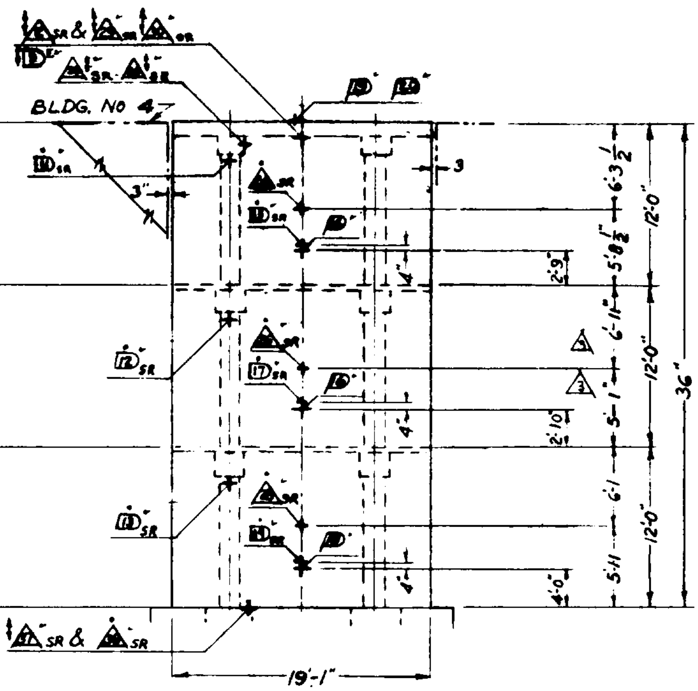
Approved: Bruce D Jones
Project 3.1 Officer

By Boyd G Anderson

[redacted]


BLDG. NO 4
ROOF
3RD FLOOR
2ND FLOOR
1ST FLOOR
36'-0"
12'-0"
19'-1"
FRONT ELEVATION
REAR ELE
ON FOUNDATION
4'-3 1/2"
10'-6
5'-3"
5"
52'-0"
17'-0"
15'-1 1/2"
8'-6"
7'-6 3/4"
2'-4 1/2"
PLAN

[redacted]



REAR ELEVATION

A^E - 1, 2	C	5165
D^E 6, 7, 8	A	5165
D^E 9, 10	D	5165
P- 19, 20, 21, 22	E	5165
A_{SR} - 37, 38, 39, 40	B	5163
A_{SR} - 33, 34, 35, 36,	B	5165
A_{SR} - 30, 31	D	5165
A_{SR} - 29, 32	C	5165
A_{SR} - 23, 24, 25, 26, 27, 28, R[illegible]	B	5164
A_{SR} - 20, 21, 22	A	5165
D_{SR} - 14, 15, 16, 17, 18, 19	B	5164
D_{SR} - 11, 12, 13	A	5164
BLDG INSTRUMENTS	DET	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

Approved: Bruce D Jones
Project 3.1 Officer

By Boyd G. Anderson

SCALE: 1/8" = 1'-0"

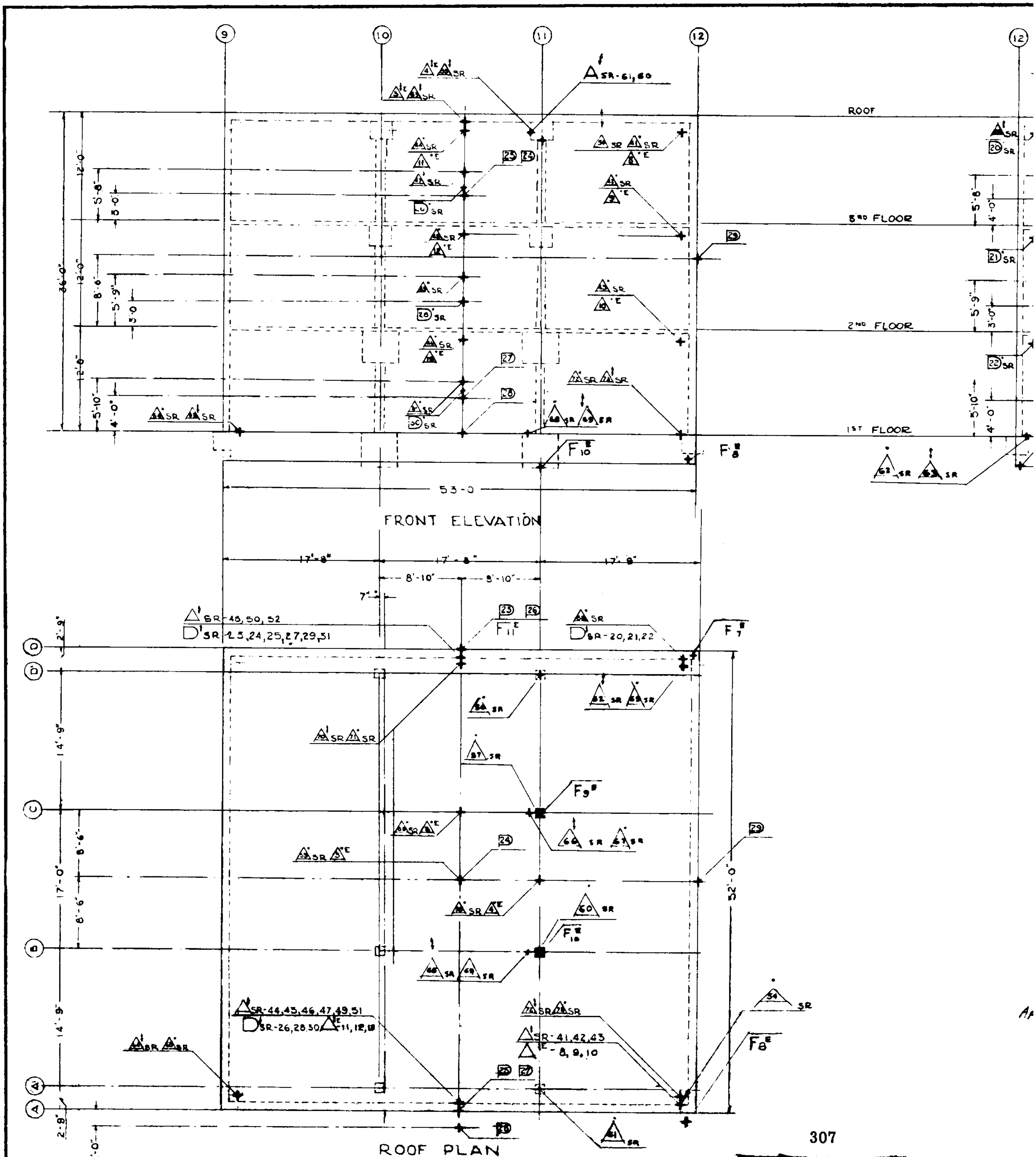
REVISIONS		
NO.	DATE	DESCRIPTION
1	5/15/50	changed 3' dim. to 4' on front & rear elevations
2	[illegible]	CORRECTED NOTATIONS
A	5/8/51	DIMENSIONS CHANGED AS BUILT AS SHOWN

HOLMES & NARVER
INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

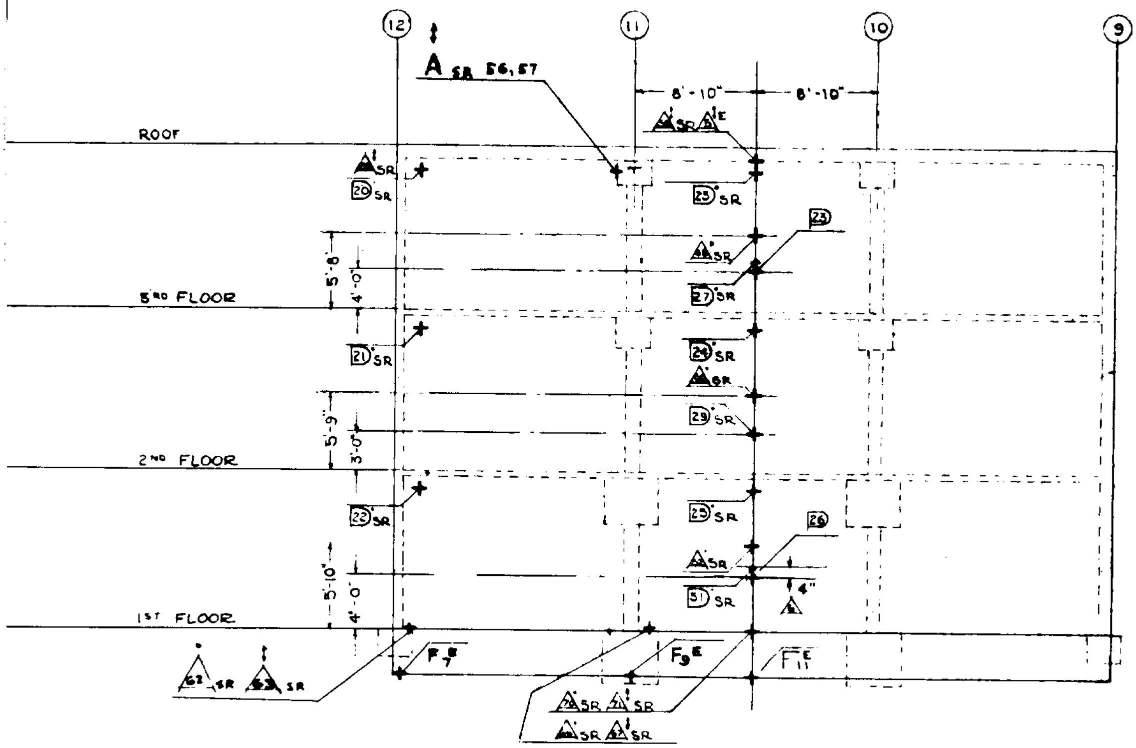
BLDG 3.1.1.(b)

DR.	CK.	DATE	SHEET NO.
R.A.P.	L.Y.	3-30-50	5153
840F	[illegible]		

[redacted]


FRONT ELEVATION
ROOF
3RD FLOOR
2ND FLOOR
1ST FLOOR
53-0
36'-0"
12'-0"
5'-8"
5'-0"
8'-6"
5'-9"
3'-0"
5'-10"
4'-0"
ROOF PLAN
17'-8"
8'-10"
7"
14'-9"
17'-0"
8'-6"
2'-9"
2'-8"
2'-0"
52'-0"
F_{11}^{E}
F_{7}^{E}
F_{9}^{E}
F_{10}^{E}
F_{8}^{E}
SR-48,50,52
SR-25,24,25,27,29,31
SR-20,21,22
SR-44,45,46,47,49,51
SR-26,28,30
11,12,13
SR-41,42,43
8,9,10
SR-61,60

[redacted]



REAR ELEVATION

D - 24	C	5167
F^E 7, 8, 9, 10, 11	—	5018
D - 29	B	5167
D - 28	A	5167
$\triangle_{SR}$ - 70, 71, 72, 73, 62, 63, 64, 65	C	5165
$\triangle_{SR}$ - 66, 67, 68, 69	B	5165
$\triangle_{SR}$ - 56, 57, 60, 61	B	5165
$\triangle_{SR}$ - 47, 48, 49, 50, 51, 52	B	5164
D - 23, 25, 26, 27	B	5164
D_{SR} - 26, 27, 28, 29, 30, 31	B	5164
$\triangle^E$ - 4; $\triangle_{SR}$ - 59	C	5166
$\triangle^E$ - 5, 5; $\triangle_{SR}$ - 55, 58	B	5166
D_{SR} - 20, 21, 22, 23, 24, 25	A	5166
$\triangle^E$ - 8, 9, 10, 11, 12, 13	A	5166
$\triangle_{SR}$ - 41, 42, 43, 44, 45, 46, 53, 54	A	5166
INSTRUMENT	DETAIL	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

REVISIONS		
NO.	DATE	DESCRIPTION
1	5-18-50	Relocated F^E mounts; No. F^E 7, 8, 9, 10
2	8-9-50	GENERAL REVISION
3	9-21-50	CHANGED D^E 11 TO 16 TO $\triangle^E$ 8 TO 13
4	9-22-50	ADDED D-24 TO TITLE BLOCK
5	3/23/51	ADDED $\triangle$ No. 54
$\triangle$	5/8/51	DIMENSION ADDED AS BUILT AS SHOWN

HOLMES & NARVER
INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

BLDG 3.1.1.(4)

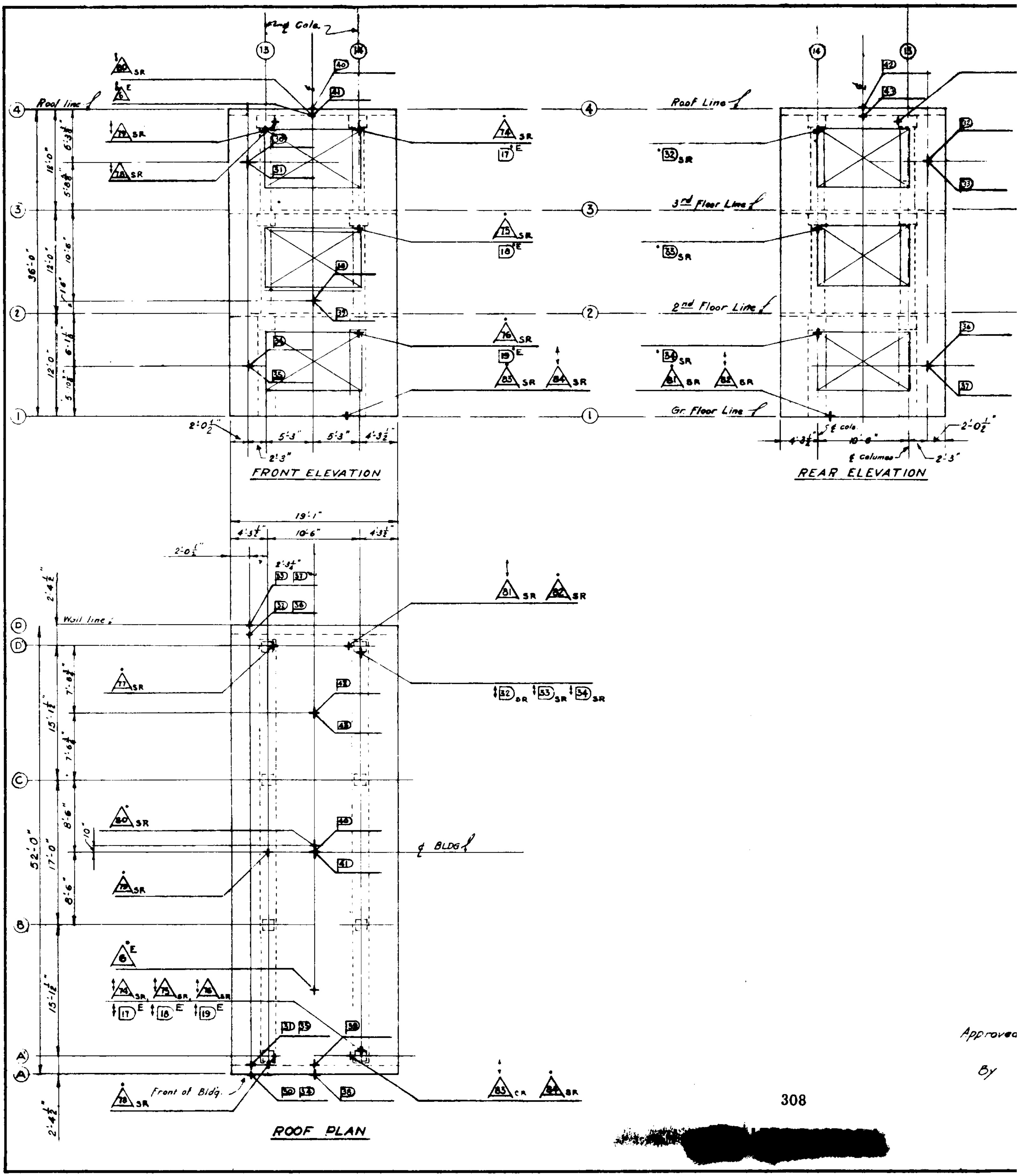
DR. JAM | CK. | DATE 8-29-50 | SHEET NO. 5156
JOB NO. | OF.

Approved: Bruce D Jones
Project 3.1 Officer

By Boyd G Anderson

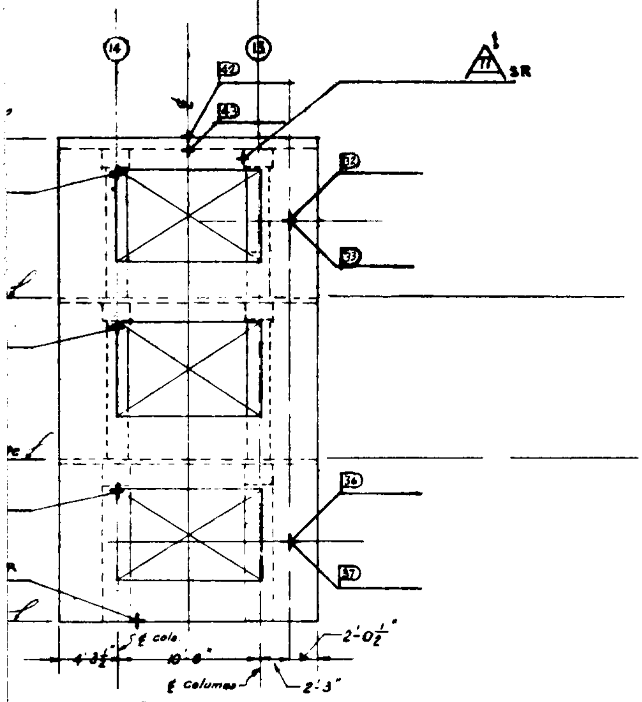
SCALE: 1/8" = 1'0"

[redacted]



FRONT ELEVATION
REAR ELEVATION
ROOF PLAN
Roof line
Roof Line
3rd Floor Line
2nd Floor Line
Gr. Floor Line
Wall line
Front of Bldg.
BLDG
19'-1"
4'-3½"
10'-6"
52'-0"
36'-0"
Approved
By

[redacted]



REAR ELEVATION

BUILDING NO. 3.1.1 (5)			
	D_{SR} 32,33,34 D^{L} 17,18,19 Δ_{SR} 74,75,76	A	5165
	Δ_{SR} 77,78	B	5165
	Δ_{SR} 79	A	5169
	Δ_{6R} 80	B	5169
	Δ_{SR} 81,82,83,84	B	5168
	30,31,32,33,34,35,36,37,38,39	A	5168
	40, 41, 42, 43	B	5168
	Δ^{E} 6	C	5169
	INSTRUMENT	DETAIL	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

REVISIONS		
NO.	DATE	DESCRIPTION
1	[illegible]	CORRECTED NOTATIONS
1	5/6/51	AS BUILT AS SHOWN

HOLMES & NARVER
INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

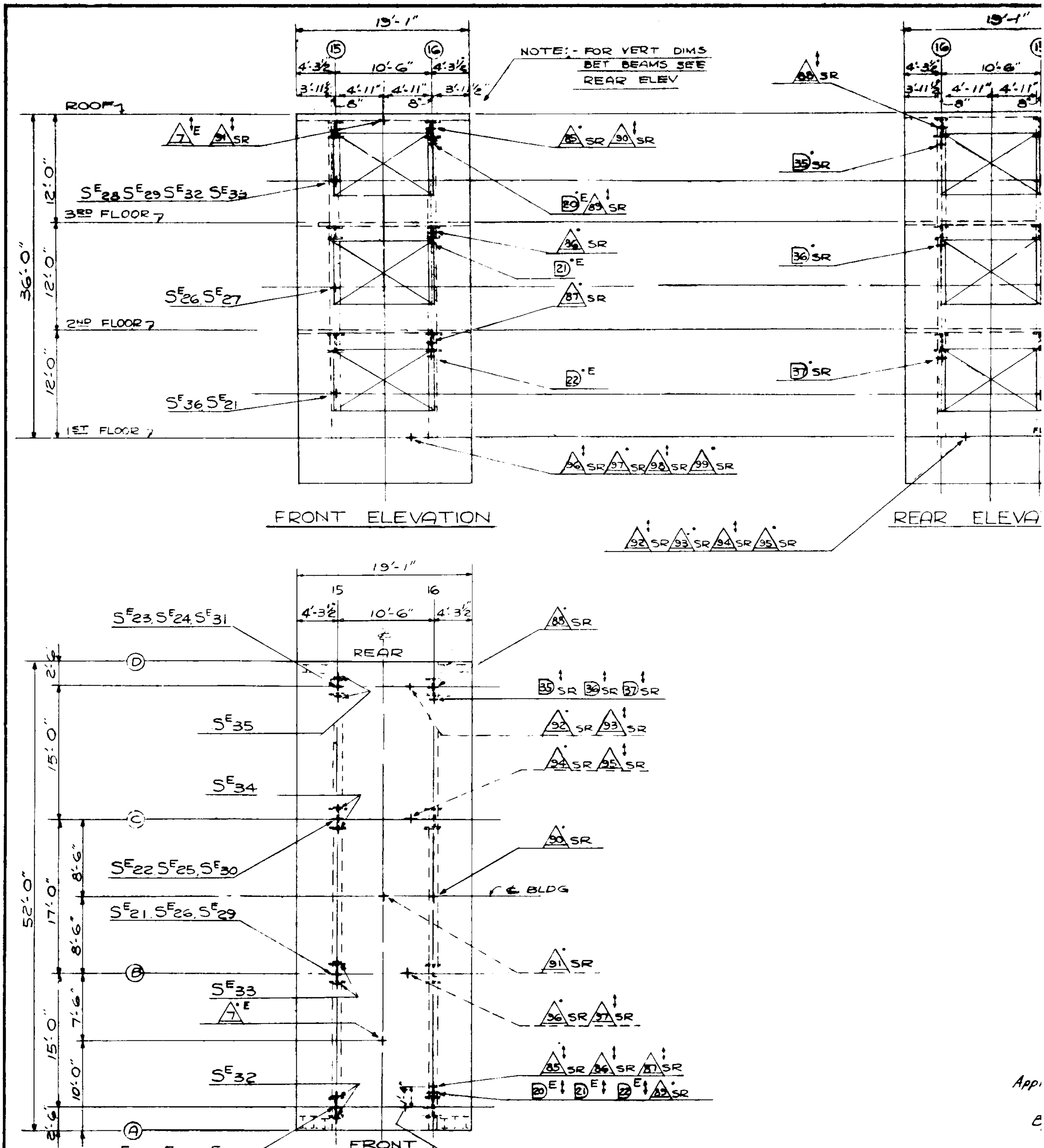
BLDG. 3.1.1(5)

DR.	CH.	DATE	SHEET NO.
[illegible]	[illegible]	3-29-50	5165
JOB NO. 840 F			

Approved Bruce D. Jones
Project 3.1 Officer

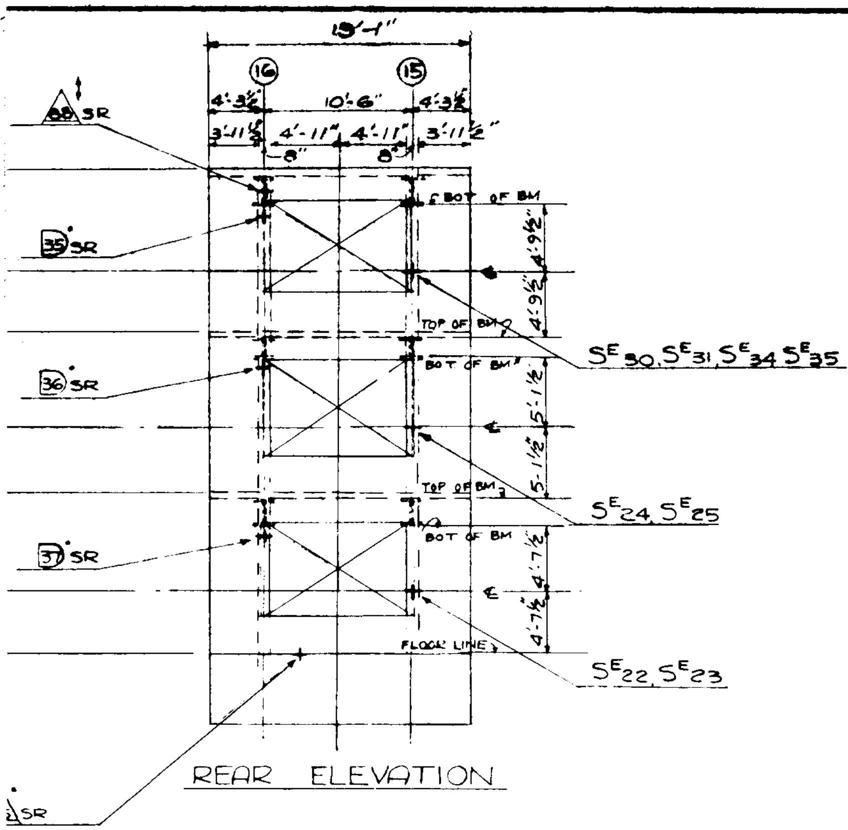
By Boyd G. Anderson

[redacted]



NOTE:- FOR VERT DIMS BET BEAMS SEE REAR ELEV
19'-1"
4'-3½"
10'-6"
ROOF
3RD FLOOR
2ND FLOOR
1ST FLOOR
36'-0"
12'-0"
FRONT ELEVATION
REAR ELEVA
REAR
FRONT
ROOF PLAN
52'-0"
15'-0"
17'-0"
8'-6"
7'-6"
10'-0"
2'-6"
℄ BLDG

[redacted]



△SR 91	D	5159
D SR-35, 36, 37	A	5162
△SR-85, 86, 87	B	5162
△SR-88, 89,	D	5162
△SR-90	C	5162
△SR-92, 93, 94, 95, 96, 97, 98, 99	A	5163
D E-20, 21, 22	A	5162
△E-7	C	5169
S E-21-36	—	—
INSTRUMENT	DET.	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

REVISIONS

NO.	DATE	DESCRIPTION
1		changed △SR to △E
2		Changed S^E to S^E36
		AS BUILT AS SHOWN

HOLMES & NARVER, INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

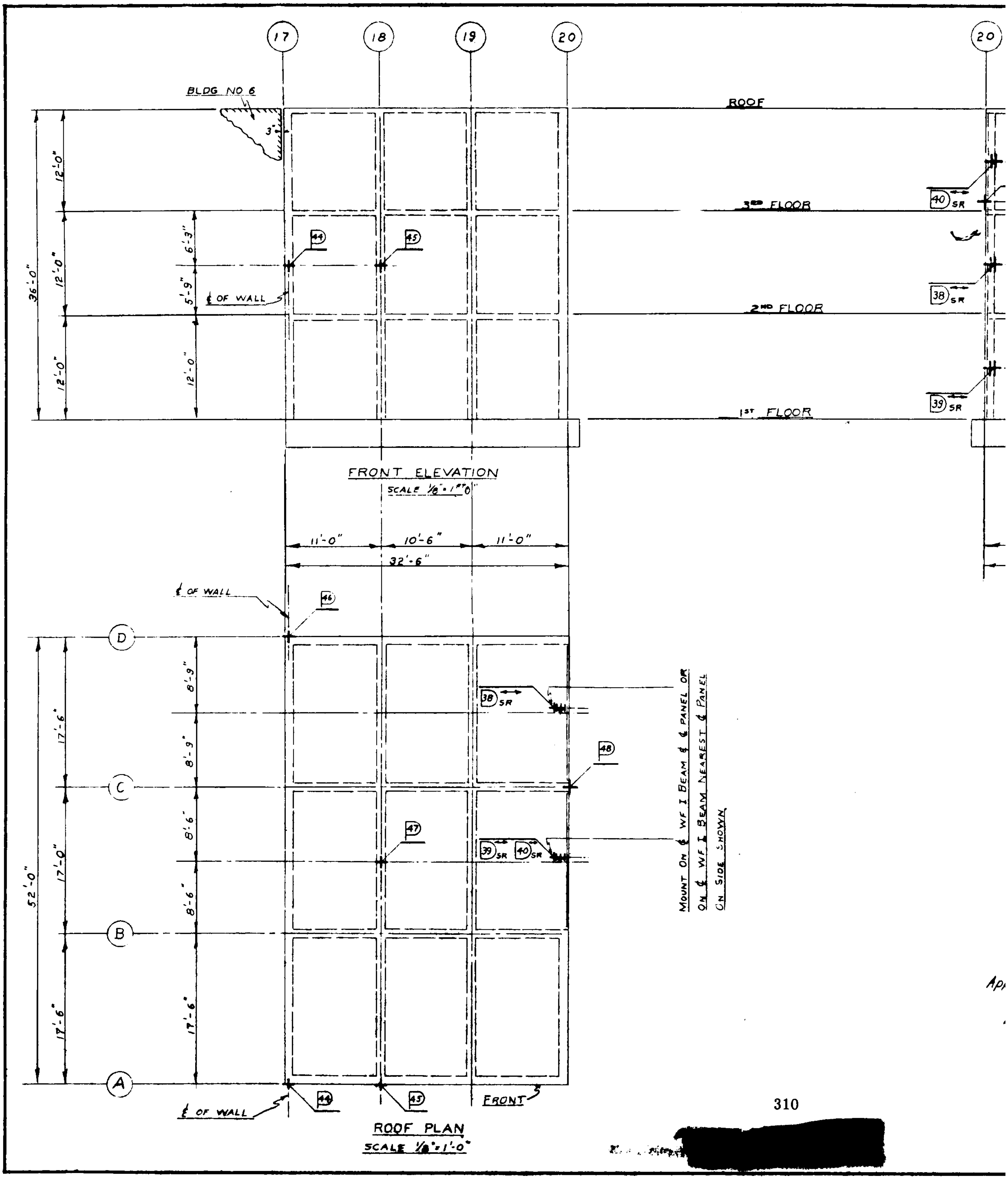
BLDG 3.1.1. (G)

F.C. | 840F | 2/3/50 | 5156

Approved: Bruce D Jones
Project 3.1 Officer

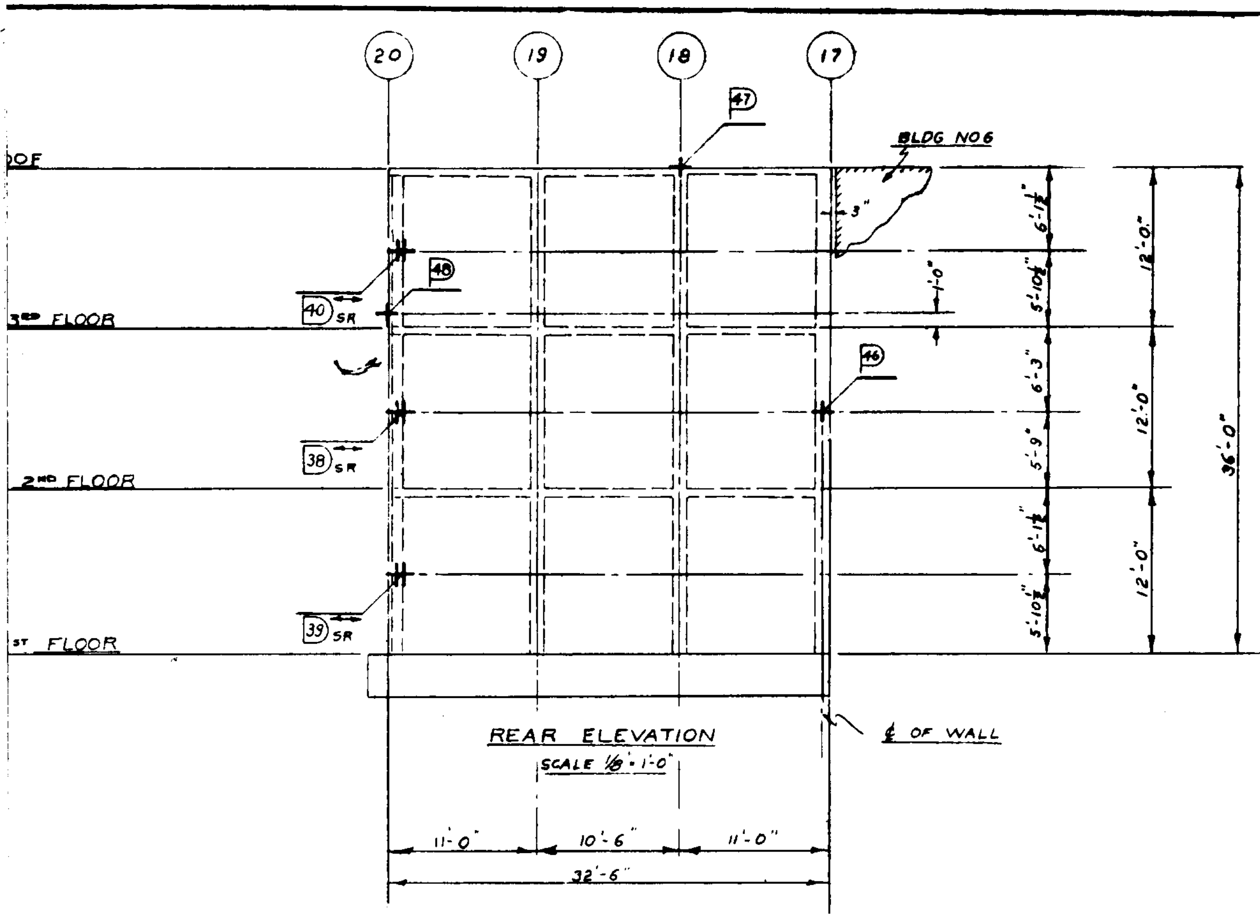
By Boyd G Anderson

[redacted]



17
18
19
20
20
BLDG NO 6
3"
ROOF
3RD FLOOR
2ND FLOOR
1ST FLOOR
36'-0"
12'-0"
12'-0"
12'-0"
6'-3"
5'-9"
12'-0"
℄ OF WALL
44
45
40 SR
38 SR
39 SR
FRONT ELEVATION
SCALE 1/8"=1'-0"
11'-0"
10'-6"
11'-0"
32'-6"
℄ OF WALL
46
D
C
B
A
52'-0"
17'-6"
17'-0"
17'-6"
8'-9"
8'-9"
8'-6"
8'-6"
17'-6"
38 SR
48
47
39 SR
40 SR
MOUNT ON ℄ WF I BEAM & ℄ PANEL OR
ON ℄ WF I BEAM NEAREST ℄ PANEL
ON SIDE SHOWN
44
45
FRONT
℄ OF WALL
ROOF PLAN
SCALE 1/8"=1'-0"

[redacted]



REAR ELEVATION

SCALE 1/8" = 1'-0"

D - 45, 46, 47, 48	B	5160
D - 44	A	5160
DSR - 38, 39, 40	C	5160
INSTRUMENT	DETAIL	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

Approved: • Bruce D Jones
Project 3.1 Officer

By Boyd G Anderson

REVISIONS

NO.	DATE	DESCRIPTION
1	5/15/57	Reversed Det A & B.
2	3/20/57	Deleted Δ 100, 101, and 102
	7/16/57	AS BUILT AS SHOWN

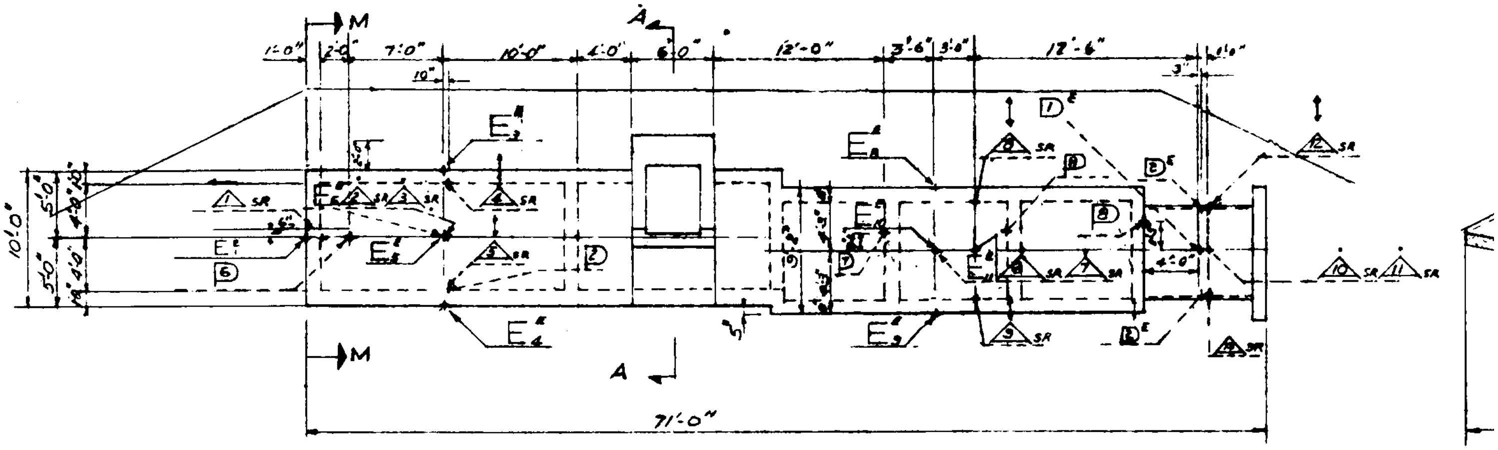
HOLMES & NARVER
INCORPORATED
ENGINEERS
824 S. FIGUEROA ST. LOS ANGELES

BLDG 3.1.1 (7)

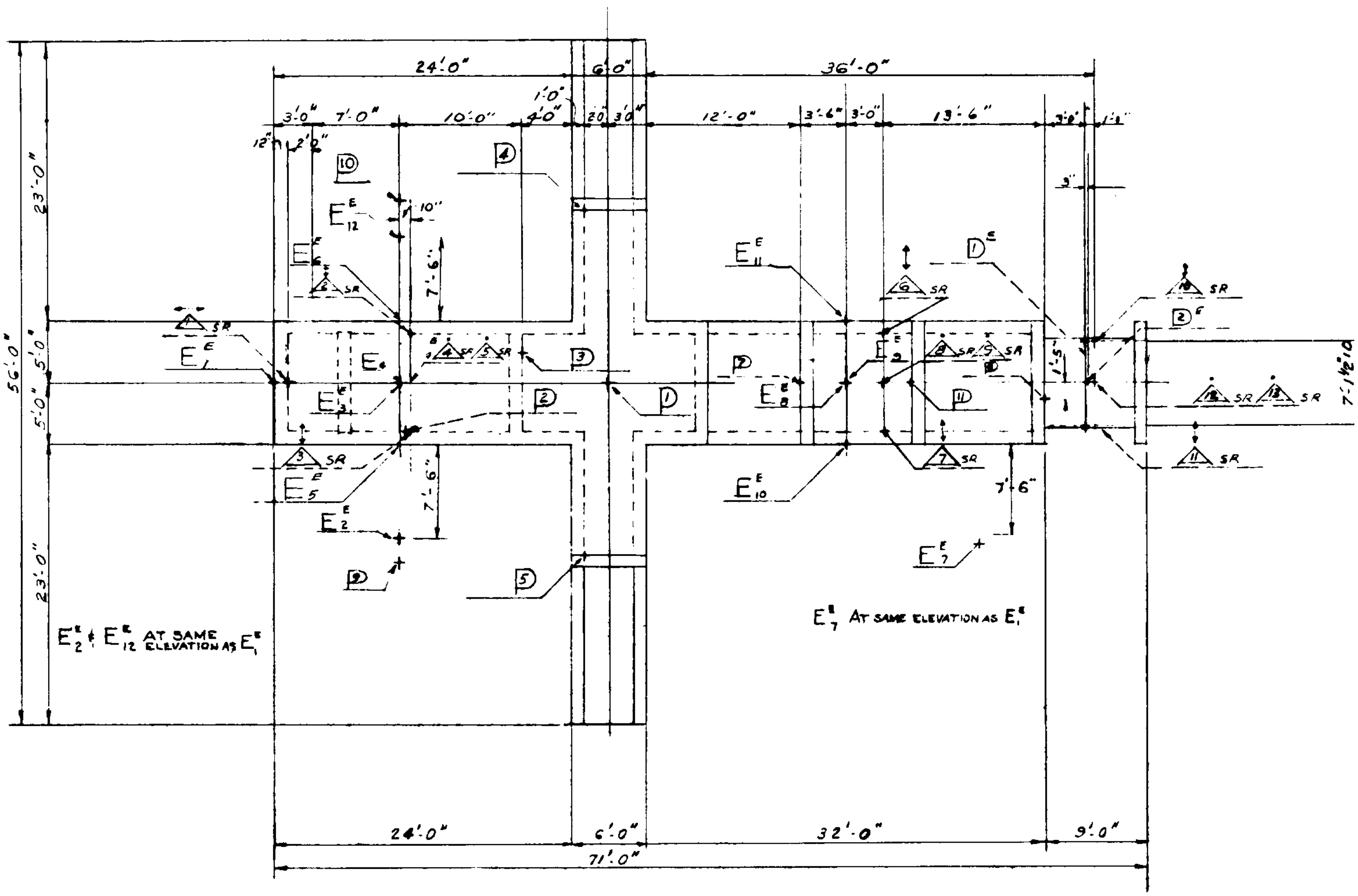
840F

5157

[redacted]



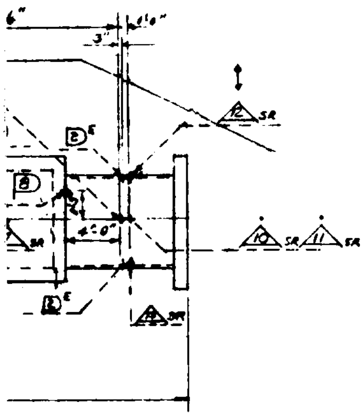
SIDE ELEVATION

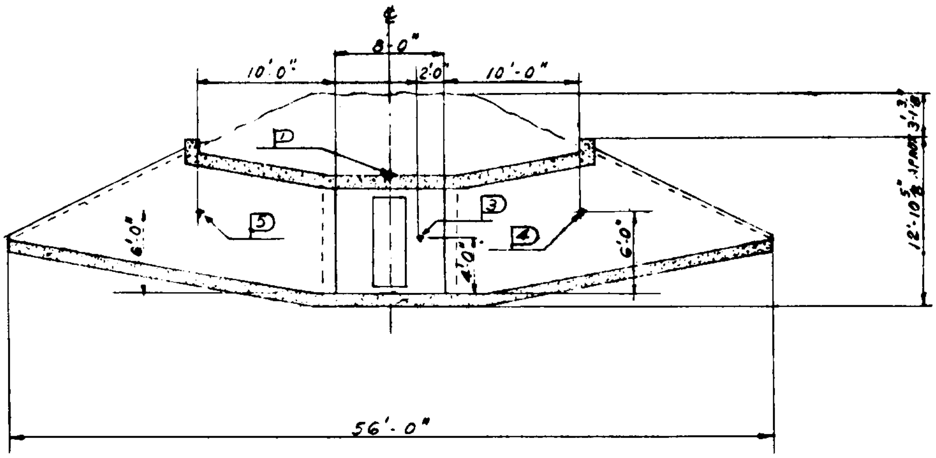


PLAN

SCALE 1/8" = 1'-0"

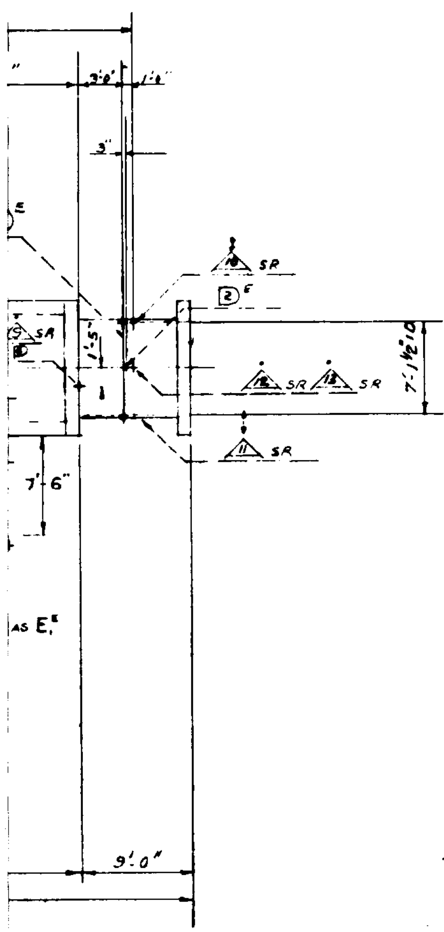
[redacted]





SECTION A-A

FOR VIEW M-M SEE DWG 5171



E^E-8, 9, 10, 11	G	5172
E^E-3, 4, 5, 6	C	5170
D^E-2	L	5173
D^E-1	J	5173
P-9, 10	M	5171
P-8.	I	5172
P-7, 11	F	5172
P-1, 3, 4, 5.	E	5171
Δ_{SR}-10, 11, 12, 13.	K	5173
Δ_{SR}-6, 7, 8, 9.	H	5172
Δ_{SR}-2, 3, 4, 5., P-2.	D	5171
Δ_{SR}-1., E^E-1.	A	5170
INSTRUMENT	DETAIL	SHEET

LOCATION OF INSTRUMENT MOUNTINGS

Approved Bruce D Jones
Project 3.1 Officer

By Boyd C A nderson

REVISIONS

NO.	DATE	DESCRIPTION
1	5/15/51	Corrected location of PD, PD, PD
2	3/23/51	Relocated E^E & Δ_{SR} 1, 2, 3, 4 & 5
3	[illegible]	NOTATIONS ADDED AS BUILT AS SHOWN

HOLMES & NARVER
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ENGINEERS
[illegible] S. FIGUEROA ST. LOS ANGELES

BLDG 315

5158

311–312

[redacted]

A P P E N D I X 5

SAMPLES OF DETAILED COMPUTATIONS

FOR DESIGN AND ANALYSIS

CONTENTS

[redacted]

Appendix 5 Cont.

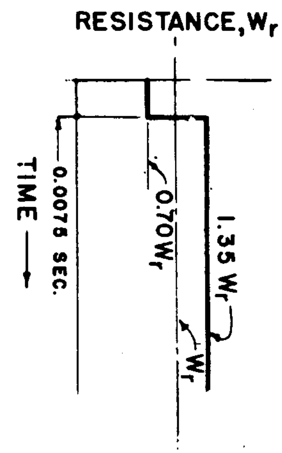
FIND MAX. ALLOWABLE BLAST LOAD
(AS OF 5/2/50)
[GIVEN RESISTANCE FROM OLD DESIGN]

$L = 11.229'$

$W_{b_{67}} = 1.09\,p$

$\frac{\Delta t^2}{m} = .238 \times 10^{-3}$

A.5.1-a Concrete One-way Slab
Numerical Analysis
Stress-Time Resistance Function



ϵ	p 100%	W_b 67%	W_R	Σ	W_b 72%	W_R	Σ	W_b 77%	W/R	Σ
0	47.5									
.0025	41	48.3	16.1	30.2	51.9	18.1	33.8	55.5	15.1	37.4
50	37.5	42.8	↓	54.9	46	↓	61.7	49.1	↓	68.4
75	34	39.0		75.8	41.9		85.5	44.8		95.1
.01	31.5	35.7	35.0	76.5	38.3	35.0	86.8	41.0	35.0	101.1
	30	33.5	↓	75.0	36.0	↓	89.8	38.5	↓	104.6
	28.5	31.9		71.9	34.3		89.1	36.7		106.3
	27.5	30.5		67.4	32.8		86.9	35.0		106.3
.02	26.5	29.4		61.8	31.6		83.5	33.8		105.1
	25.5	28.3		55.1	30.5		79.0	32.6		102.7
	24.5	27.2		47.3	29.2		73.2	31.2		98.9
	23.5	26.2		38.5	28.1		66.3	30		93.9
.03	22.5	25.1		28.6	27.0		58.3	28.9		87.8
	22.0	24.3		17.9	26.1		49.6	27.9		80.7
	21.5	23.7		6.6	25.5		39.4	27.2		72.9
	20.5	22.9		-5.5	24.6		29.5	26.3		64.2
.04	20	22.1			23.8		18.3	25.5		54.7
	19.5	21.7		702.0	23.3		6.6	24.9		44.6
	18.5	20.7			22.2		-6.2	23.7		33.3
	18	19.9			21.4			22.9		21.2
.05	17.5	19.3			20.8		1033	22.2		8.4
										1488

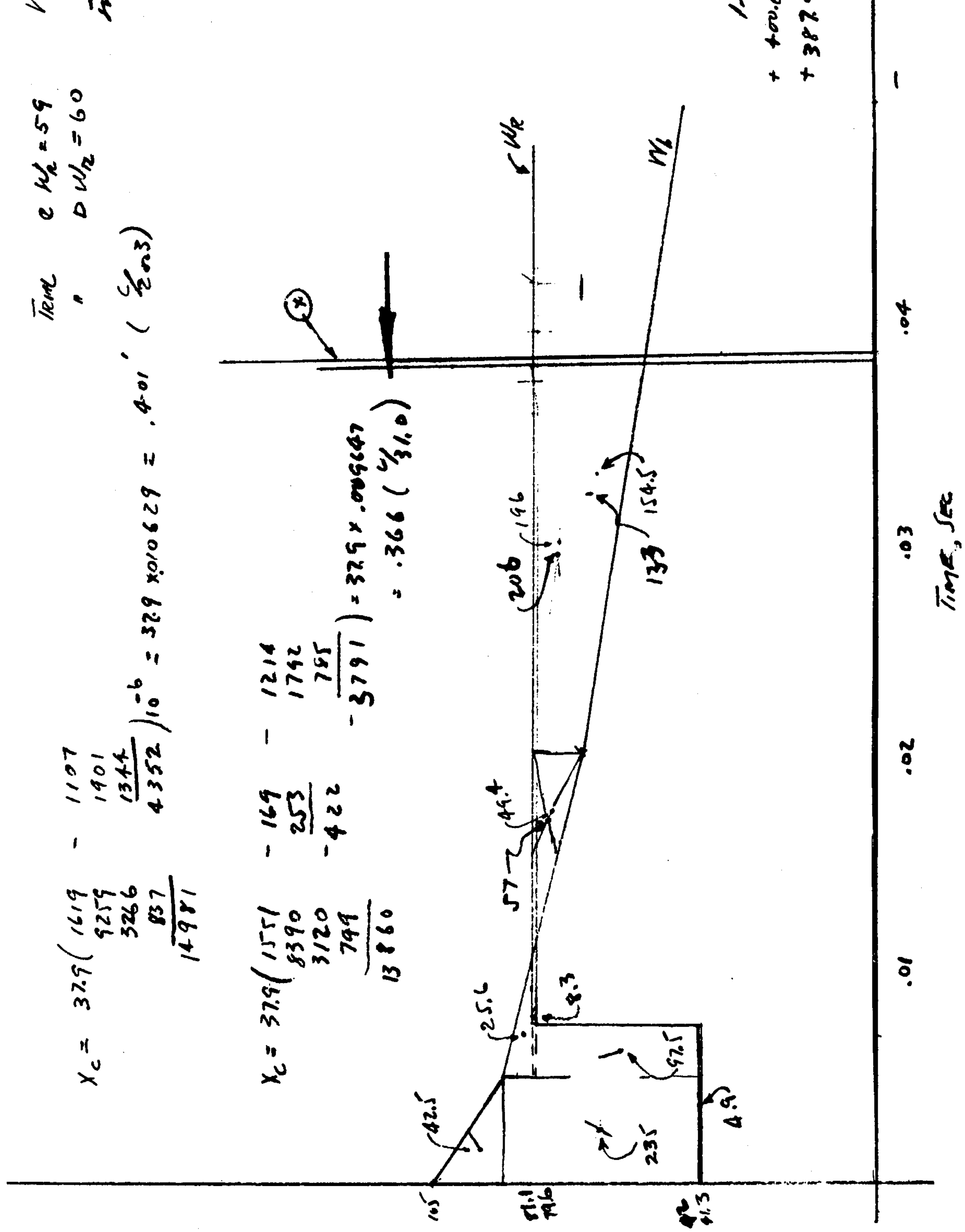
A.5.1-a

$X = .167 \left(\frac{b}{67}\right)$ $\qquad X = .246 \left(\frac{L}{46}\right)$ $\qquad X = .354 \left(\frac{L}{31.7}\right)$

$X = \left[\Sigma\Sigma(W_b + W_r)\right] \frac{\Delta t^2}{m}$

[redacted]

A5.1-b Concrete One-way Slab
Semi-Graphical Analysis
Stress-Time Resistance Function



A.5.1-b FORCE, KIPS

[redacted]

See also MM's 61a of 4/6/50 & ff

A.5.1-c Concrete One-way Slab Numerical Analysis Stress-Strain Resistance Function

Span $= 15.17'$ $t = 11''$ $1''\phi$ @ $10''$ $A_s = 0.95$

$a = 1.55$

$M_R = 415''^{K}$

$135 M_R = \frac{415 \times 1.35}{12} = 46.6^{KF}$

$W_b = 2.94p$ (@135%)

$W_s = 1.73^{K}$

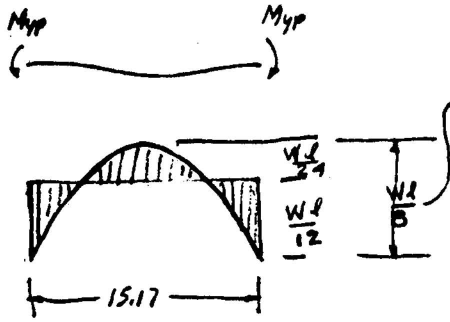
$I = \frac{\cancel{12} \times 11 \times 11 \times 11}{\cancel{12}} = 1331$

$m_{pl} = 0.0447$ $(m_{pl} = \frac{2}{3} m)$

$\frac{1}{m_{pl}} = 22.4$

$\frac{1}{m_f} = 37.4$ $(m_f = 0.4\ m)$

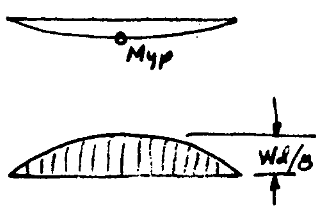
$\frac{1}{m_s} = 29.9$ $(m_s = 0.5\ m)$



$\frac{Wl}{8} = 1.5 M_{yp}$; $\frac{W_R l}{12} = M_{yp}$

$\frac{W_R l}{12} = M_{yp}$; $W_{r_1} = \frac{46.6 \times 12}{15.17} = 37.0^{K}$

$\Delta_{el} = \frac{W l^3}{384 EI} = \frac{37.0 \times 15.17^3 \times 1728}{384 \times 3000 \times 1331} = 0.145''$
$= 0.0121^{FT}$



$\frac{Wl}{8} = \frac{46.6}{2}$; $W = \frac{46.6 \times 4}{15.17} = \frac{37.0}{\underline{12.4}}$
$49.4 = W_R$ (TOT)

$\Delta_{el} = \frac{5}{384} \times \frac{12.4 \times 15.17^3 \times 1728}{384 \times 3000 \times 1331}$

$= 5 \times \frac{12.4}{37.0} \times .0121 = 0.0202^{FT}$

$\underline{0.0121}$

Δ_{el_T} 0.0323^{FT}

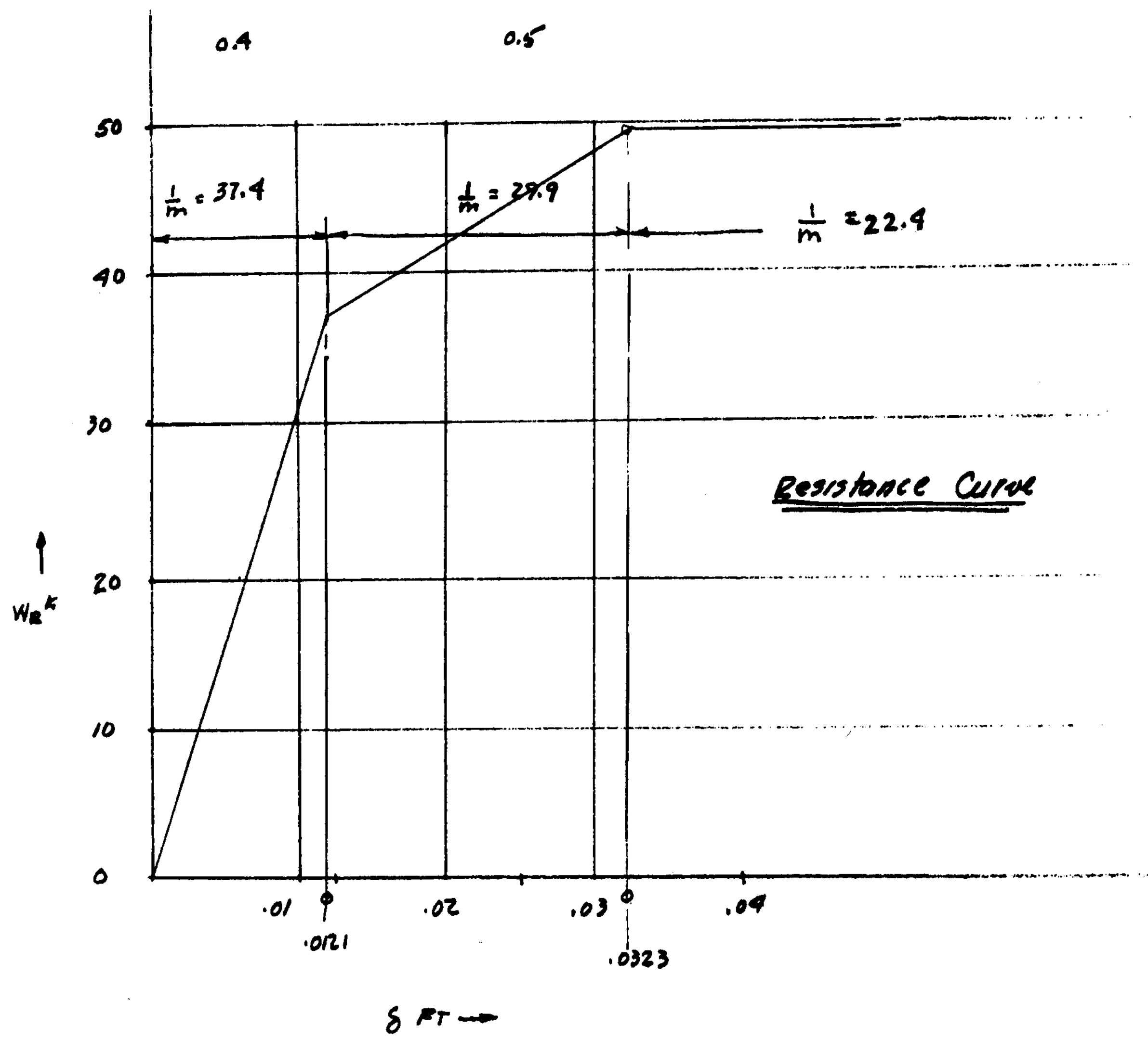
(IT IS ASSUMED IN THIS COMPUTATION THAT THE INITIAL RESISTANCE OF SLABS & GIRDERS IS ZERO; THIS IS NOT QUITE TRUE SINCE BEFORE THE BLAST HITS, IT HAS DEVELOPED ENOUGH "RESISTANCE" TO CARRY THE DEAD LOAD, BUT THE ERROR IS SMALL, AND ON THE SAFE SIDE)

A.5.1 - c

[redacted]

up to .0121 $\quad W_R = \frac{37.0}{.0121} = 3060\ \Delta^k \quad W_R = 37.0$

.0121 → .0323 $\quad \Delta W_R = \frac{12.4}{.0202} = 615\ \Delta^k \quad W_R = 49.4$



A.5.1-c

[redacted]

135% $\frac{1}{m} = 37.4$; $37.8 \times .0025 = .0935$ $\Delta W_r = 3060\Delta$ @.0121 $\frac{1}{m} = 22.4$

$\frac{1}{m} = 29.9$; $29.9 \times$ " $= .0748$ $\Delta W_r = 615\Delta$ .0323 $W_b = 2.94p$ @135%

t	p	Δt	W_p	W_s	$W_p + W_s$	W_R	F	Δv	v_0	$\bar{v}$	Δd	d	W_{r_0}	$\overline{W_r}$	
.015 .0175	0 1.875 3.75	.0025	5.5	1.7	7.2	1.3 1.1	5.9 6.1	0.55 0.57	0.55 0.57	0.27 0.28	.0007 .0007	.0007 .0007	2.1 2.1	1.1 1.1	
.0200	5.625 7.5	.0025	16.5	1.7	18.2	8 6.2	10.2 12.0	0.95 1.12	1.52 1.69	1.04 1.13	.0026 .0028	.0033 .0035	10.1 10.7	6.1 6.4	
.0225	9.375 11.25	.0025	27.6	1.7	29.3	18 19.0	11.3 10.3	1.06 0.96	2.75 2.65	2.22 2.17	.0055 .0054	.0090 .0089	27.6 27.3	19.1 19.0	
.0250	13.125 15.0	.0025	38.6	1.7	40.3	32 33 33.4	8.3 7.3 6.9	0.78 0.68 0.64	3.43 3.33 3.29	3.04 2.99 2.97	.0076 .0075 .0074	.0165 .0164 .0163	39.7 39.6 39.5	33.5 33.5 33.4	
.0275	14.875 14.75	.0025	43.6	1.7	45.3	43 42.2	2.3 3.1	0.17 0.23	3.46 3.52	3.37 3.40	.0084 .0085	.0247 .0248	44.7 44.8	42.1 42.1	
.0300	14.625 14.5	.0025	43.0	1.7	44.7	47.1	-2.4	-0.18	3.34	3.43	.0086	.0334	49.4	47.1	pl.
.040	13.9 13.3	.010	41.0	1.7	42.7	49.4	-6.7	-1.58	1.84	2.59	.0259	.0593	49.4	49.4	
.050	13.0 12.6	.010	38.2	1.7	39.9	49.4	-9.5	-2.12	-0.28	0.78	.0078	.0671			

$\delta_{max} = .0671$ FT

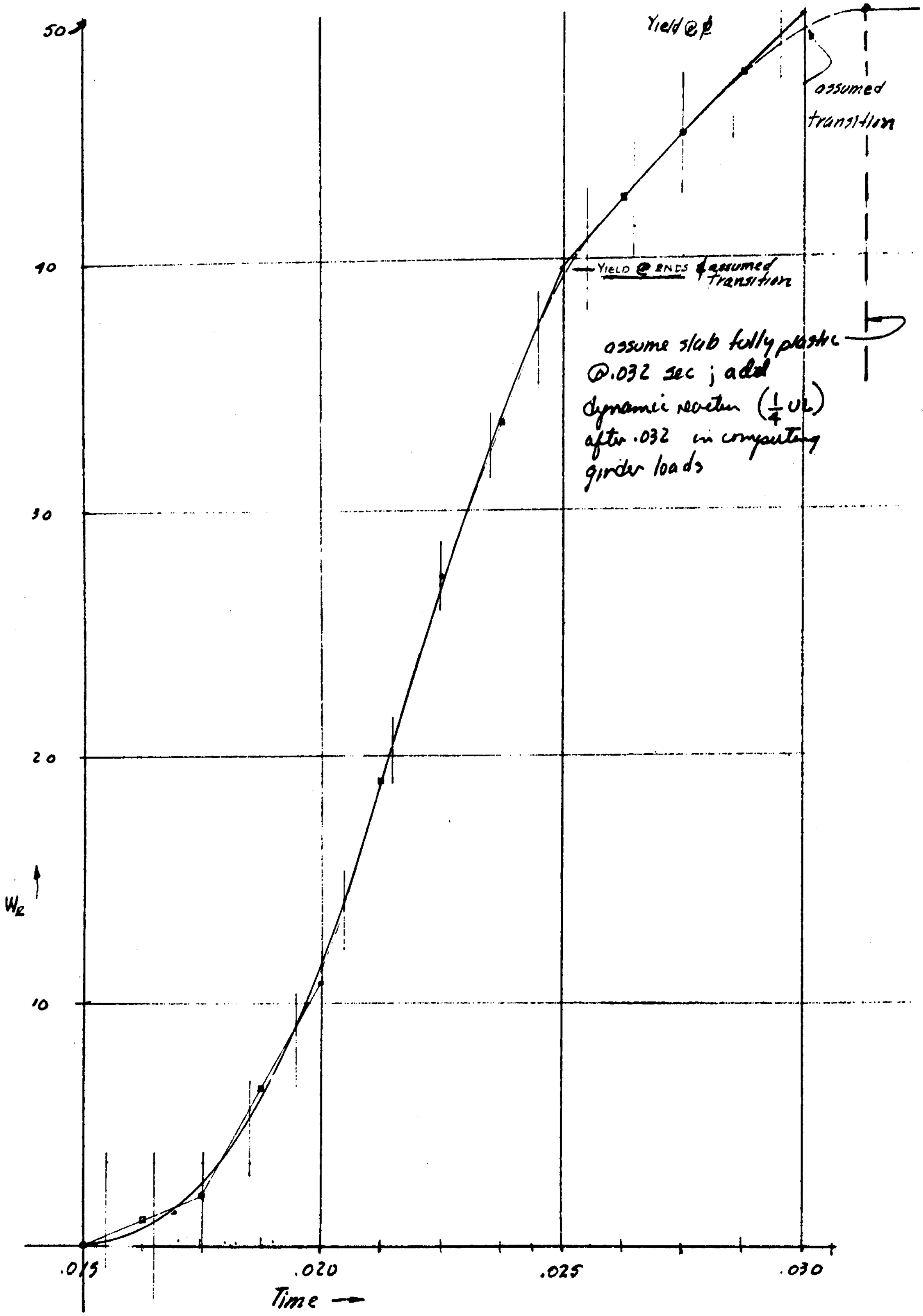
$a_{max} = +448$ FT/sec²

-232 FT/sec²

trace Vibration after rebound sets in

A.5.1-c

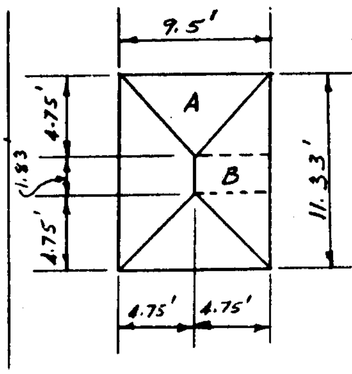
[redacted]



<u>W_R Roof Slab - Section 4 @ 135%</u>

A.5.1-c

[redacted]



SEGMENT A

Try: 12" Wall - ½"φ @ 7" o.c. - $A_s = .34^{\square''}$

$\frac{\Delta t}{m} = .0476$ (see sh. 30)

$W_B = 4.06p^k$

$W_R = 211^k$; $w_R = 147.9^k$, $w_R = 285.0^k$ } see sh. 30

A.5.2 Concrete Two-way Slab

t	P P.S.I.	W_B (4.06p) Kips	W_R	$W_B - W_R$	$\times \frac{\Delta t}{m} =$ Δv 1/sec.	v_0	$\bar{v}$	$\bar{v}\,\Delta t = \Delta x$	x (ft.)
0	59.5	241.5				0			0
		239.5	147.9	91.6	4.36		2.18	.0055	
.0025	58.5	237.5				4.36			.0055
		235.3	147.9	87.4	4.16		6.44	.0161	
.0050	57.5	233.0				8.52			.0216
		231.0	147.9	83.1	3.95		10.495	.0262	
.0075	56.5	229.0				12.47			.0478
		227.3	285.0	- 57.7	-2.74		11.10	.0278	
.0100	55.6	225.5				9.73			.0756
.0100	20.2	82.0							
		81.6	285.0	-203.4	-9.69		4.885	.0122	
.0125	20.0	81.2				0.04			.0878
		80.6	285.0	-204.4	-9.74				
.0150	19.7	80.0							
		79.3							
.0175	19.4	78.7							

$\frac{l}{64} = \frac{9.5}{64} = .1485$ .. O.K.

Use Press. Curve D

Use: 12" Wall - ½"φ @ 7" o.c. - Vert. & Horiz. - Both Faces.

for $A_s = .30$ $\Delta = .1174$

2nd Fl. - 3rd Fl.

Long.

$.30 \times 1.5 = .45$ — ½"φ @ 5½"

$.45/2 = .225$ Use .34 - ½"φ @ 7

Transv.

$3\frac{1}{2} = .30$; $x = .20$ - ½ φ 7

$2x = .40$ = ½"φ @ 6"

A.5.2

[redacted]

SEGMENT B

Try: 12" Wall - $\frac{3}{4}$"ϕ @6" o.c. - $A_s = .88^{\square}$"

$\frac{\Delta t}{m} = .0927$ } See sh. 33
$W_B = 1.564_p$

$A_s f_s = .88 \times 50{,}000 = 44{,}000^{\#}$ $\qquad a = \frac{44{,}000}{2550 \times 12} = 1.438$

$M_{yp} = (10.875 - .719) \times 44{,}000 = 10.156 \times 44{,}000 = 447^{"K}/ft.$

$M_R = 447 \times \frac{1.83}{12} \times 2 = 136.2^{'K}$ $\qquad W_R = \frac{136.2}{2.375} = 57.4^K$ $\qquad w_R = 40.2^K$, $w_R = 77.5^K$

t	P p.s.i.	1.564_p W_B Kips	W_R Kips	$W_B - W_R$	$\times \frac{\Delta t}{m} = \Delta V$	V_0	$\bar{V}$	$\bar{V} \times \Delta t = \Delta x$	x (ft.)
0	59.5	93.1				0			0
		92.3	40.2	52.1	4.83		2.415	.0060	
.0025	58.5	91.5				4.83			.0060
		90.7	40.2	50.5	4.68		7.17	.0179	
.0050	57.5	89.9				9.51			.0239
		89.1	40.2	48.9	4.53		11.775	.0294	
.0075	56.5	88.4				14.04			.0533
		87.6	77.5	10.1	0.94		14.51	.0363	
.0100	55.6	86.9				14.98			.0896
.0100	20.2	31.6							
		31.4	77.5	-46.1	-4.27		12.845	.0321	
.0125	20.0	31.3				10.71			.1217
		31.1	77.5	-46.4	-4.30		8.56	.0214	
.0150	19.7	30.8				6.41			.1431
		30.6	77.5	-46.9	-4.35		4.235	.0106	
.0175	19.4	30.4				2.06			.1537

$\frac{l}{64} = .1485$ ∴ Say O.K. -

Use: 12" Wall - $\frac{3}{4}$"ϕ @6" o.c. - Horiz. } Both Faces
$\frac{1}{2}$"ϕ @7" o.c. - Vert.

A.5.2

[redacted]

80% OF BLAST 12WF27 Girts @ 3'-5½" o.c.

SPAN = 41½" − 3½ = 38" **A.5.3** **V-Beam Panel**

$Dyn\ M_{yy} = 22.8''^{K}$ $\quad w_R = \frac{16 \times 22.8}{38} = 9.6^{K}$

$w_1 = \frac{12 \times 22.8}{38} = 7.2^{K}$

$$\Delta elastic = \frac{(38)^3}{384 \times 3 \times 10^4 \times .312} \left(5 \times 9.6 - 4 \times 7.2\right) = .292'' = .0243'$$

19.2

$w_B = .80\, p\, \ell\, b = .80 \times \frac{12}{1000} \times 38\ p = .365\ p^{K}$

$m = \frac{1}{2} \times \frac{.0034^{k}/'}{32.2} \times \frac{38}{12} = .000167\ ^{ksf}/_{ms^2}$

$\Delta v = \frac{P \Delta t}{.000167}$; $\quad w_{RF} = \frac{9.6}{.0243} x = 395x$

t	Δt	$w_D = .365\,p$	Assumed W_R	P	Δv	v	$\bar{v}$	Δx	x	W_{RF}
.0005	.0005	6.14	1.14	5.00	14.95	14.95	7.475	.00374	.00374	1.43
			.84	5.30	15.85	15.85	7.925	.00396	.00396	1.57
			.80	5.34	16.00	16.00	8.000	.00400	.00400	1.58 ←
.0010	.0005	6.14	3.14	3.00	8.98	24.98	20.49	.01025	.01425	5.64
			3.48	2.66	7.98	23.98	19.99	.00999	.01399	5.52
			3.54	2.60	7.78	23.78	19.89	.00995	.01395	5.51 ←
.0015	.0005	6.12	7.12	−1.00	−2.95	20.83	22.30	.01115	.02510	7.71 ave
			7.62	−1.50	−4.48	19.30	21.54	.01077	.02472	7.63 ←
.00242	.00092	6.10	9.60	−3.50	−19.30	0	9.65	.00888	.03360	9.60

Stops @ .00242 Sec $\quad \Delta = .03360'$

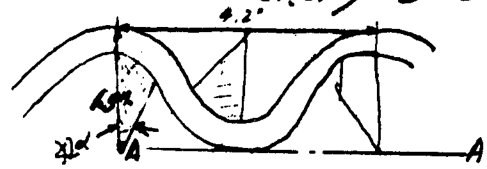
Δ ALLOWABLE $= \frac{38}{32} = 1.185'' = .0986'$

A.5.3

[redacted]

Det Section Modulus of 1 Corrugation
see Diag. Pg. BMT-3 of Johns-Manville Catalog DS-300

Take Mom. of Inertia about Line A-A



$I_{\square} = \frac{bd^3}{3} = \frac{4.2(1.5)^3}{3} = \underline{4.72}''^4$

$I_{\cap\, c.g.} = \frac{R^4}{4}\left[\alpha + \sin\alpha\cos\alpha - \frac{16\sin^2\alpha}{9\alpha}\right]$

$= \frac{(1.062)^4}{4}\left[.353 + .346(.938) - \frac{16}{9}\frac{(.346)^2}{.353}\right]$

$= .024''^4$

$A_{\cap} = \alpha R^2 = \frac{.705}{2}(1.062)^2 = .398''^2$

$A_{\cap}\bar{x}^2 = .398(.695)^2 = .192''^4$

$\bar{x} = 2R\frac{\sin\alpha}{3\alpha} = 2(1.062)\frac{.346}{3(.353)} = .695''$

$\sin 2\alpha = \frac{.69}{1.063} = .648$

$\cos 2\alpha = .762$

$2\alpha = .705$ rad $= 40°-23'$

$\alpha = 20°-12'$

$\sin\alpha = .346$

$\cos\alpha = .938$

$\alpha = .353$

$I_{\cap AA} = .192 + .024 = \underline{.216}''^4 \times 2 = \underline{.432}''^4$

A.5.4. Transite Panel

c.g. $\cap$ on upper curves $= 1.5'' - .695'' = .805''$

$I_{\cap AA} = .024 + .398(.805)^2 = .262''^4 \times 2 = \underline{.524}''^4$

$I_{\Delta AA} = \frac{bd^3}{12} = \frac{1.8(.8)^3}{12} = .077''^4 \times 2 = \underline{.154}''^4$

upper Δ: $I_{\Delta\, cg} = \frac{bd^3}{36} = .0256''^4$

$A_{\Delta} = \frac{1}{2}bd = .72 \qquad \bar{y} = 1.234''$

$\frac{1}{3}(.1)(.4) = .0133$

$I_{\Delta AA} = .026 + .72(1.234)^2 = 1.108''^4 \times 2 = \underline{2.216}''^4$

Correction for approx dimensions of Δ: $A_{small\,\Delta} = -.013''^2$ - dim. too small

$I_{small\,\Delta} = 2\left[.013'' (1.5'')^2\right] = -.061''^4$

I_{AA} Total: $4.720 - .432 - .524 - .154 - 2.216 = 1.394''^4 - .061 = 1.333''^4$

Area Total $= 6.3'' - 4\left(\frac{.795}{2}\right) - 4(.72) = 1.83''^2$

$I = 1.83(.75)^2 = 1.03''^4 - 1.33 = .30''^4$

S.M. $= \frac{.30}{.75} = .400''^4$ should be $.354''^3$

A.5.4

[redacted]

End B, 1F → 2F, Design for 100% of Blast

$M_{yp} = S\frac{I}{C} = 4140\ psi \times .354\ ''^3/corrug. = 1470\ ''\text{-}\#/corrug$

$M_{yp} = \frac{1}{12}\, w\, \ell^2\ ;\ 1470\ ''\text{-}\# = \frac{1}{12}(2 \times 20\ psi)(4.2\ ''/corrug)\ \ell^2$

$\ell = 10.45'' =$ nec. spacing of girts to stop deflection elastically assuming R.F. = 2

Try $\ell = 10.5''$ spacing of girts

$M_{yp}/ft = S\frac{I}{c} = 4140\ psi \times .354\ ''/corrug \times \frac{12''}{4.2''} = 4190\ ''\text{-}\#/'$

$1.5\, M_{yp} = \frac{W_e \ell}{8}\ ;\ 1.5(4190\ ''\text{-}\#/') = \frac{W_e}{8}(10.5'')\ ;\ W_e = 4790\ \#/'$

$W_B = 10.5''(12'')(20\ \#/in^2) = 2520\ \#/'$

$m_s = \frac{10.5''(12'')}{144\ ''/'} \times 4.1\ \#/'^2 \times \frac{1}{32.2} \times \frac{1}{2} = .0556\ \#/'/sec^2$

$\Delta_{el} = \Delta_{all} = \frac{W_e \ell^3}{384\, EI} = \frac{4.79^k (10.5'')^3}{384(\frac{30}{2} \times 10^3\ k/in^2)(.76\ ''^4)} = .00951'' = .000791'$

τ	W_B	W_e	P_N	v	$\bar{v}$	x	W_{RF}
0				0		0	
	2520	160	2360		1.06		
.00005				2.13		.000053	322
	2520	765	1755		2.92		
.0001				3.71		.000199	1205
	2520	1780	740		4.05		
.00015				4.38		.000401	2430
	2520	3040	-520		4.15		
.0002				3.91		.000607	3700
	2520	4170	-1650		3.17		
.00025				2.43		.000768	4640
	2520	4790	-2270		1.41		
.0003				.39		.000838	4790
	2520	4790	-2270				
.00035				-1.65			

$\Delta_{Total} = .000838'$

$\Delta_{all} = .000791'$

Deflection 6% high

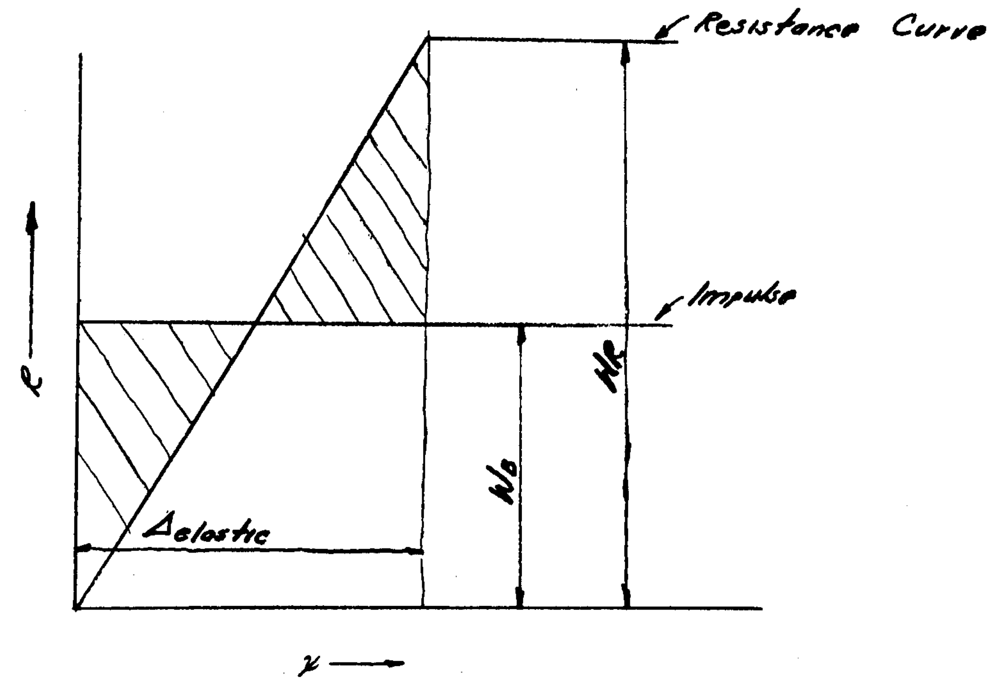
ok.

Note: This proves 10.5" spacing is good for R.F. = 2

A.5.4.

[redacted]

Check on use of Response
Factor of 2



Note: For very small time interval, the load impulse can be considered constant.

A.5.4

[redacted]

Interior Span

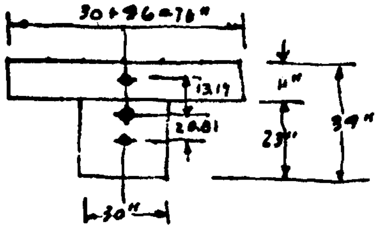
Span = 15.5' ÷ 4 = 3.88' = $46\frac{1}{2}$"

FOR I, USE (ASSUME) THIS SECTION:

A.5.5 **Concrete Girder - Plastic**

Numerical Analysis

Stress-Strain Resistance Function



eq.

$76 \times 11 = 836 \times 5.5" = 4600$

$23 \times 30 = \underline{690} \times 11.5 = \underline{7940}$

$1526 \qquad 3340$

$\bar{y} = \dfrac{3340}{1526} = 2.19"$

$I = \dfrac{76 \times 11^3}{12} = 8430$ (1331)

$+ 836 \times 7.69^2 = 49\,500$

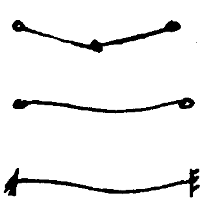
$+ \dfrac{30 \times 23^3}{12}$ (12200) $\quad 30418$

$+ 690 \times 13.69^2 = \dfrac{129\,600}{217{,}348}$ in.^{4} $\left[\dfrac{bd^3}{12} = 34^3 \times \dfrac{30}{116}^{2.5} = 98{,}260 \text{ (wo. flange)}\right]$

From MMS P.62

$W_1 = 651^K \quad \therefore \quad \Delta_1 = \dfrac{651 \times 15.5^3 \times 1728 \times}{384 \times 3000 \times 217{,}348 \times 12} = .0140$ FT (144)

$W_2 = \dfrac{207^K}{858} \quad \therefore \quad \Delta_2 = 5 \times \dfrac{207}{651} \times .0140 \qquad \dfrac{= .0222}{.0362}$

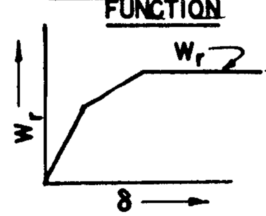


$\frac{1}{m} = 0.968 \left[\quad (m_e = .666\ m) \right.$

$\frac{1}{m} = 1.29 \left[\quad (m_e = 0.50m) \right.$

$\frac{1}{m} = 1.62 \left[\quad (m_e = 0.40m) \right.$ .00162

RESISTANCE FUNCTION



$Wr_1 = \dfrac{651}{.0140} = 46{,}500\,\Delta$ ← to Max W_E of 651 @ $\Delta = .0140$

$Wr_2 = \dfrac{207}{.0222} = 9{,}340\,\Delta$ ← to Max W_E of 858 @ $\Delta = .0362$

A.5.5

[redacted]

ϵ	ρ	Slab W_L	Slab $\frac{1}{4}UL$	W_S	W_b	W_T	W_E	F	Δw	w_0	$\bar{w}$	$\Delta\phi$	ϕ	W_r calc
.015 .016	0 (.75) 1.5	(0.5) 7.8		16.4	5.6	29.8	0	29.8	0.05	0.05	0.02	0	0	0
.017	(2.25) 3.0	(1.2) 18.6		16.4	17.0	52.0	2 2.3	52x 49.7	0.08 0.08	0.13 0.13	0.09 0.09	.0001 .0001	.0001 .0001	(4.6) 2.3 (4.6) 2.3
.018	(3.75) 4.5	(2.6) 40.4		16.4	28.2	85.0	10 9.3	75 75.7	0.12 0.12	0.25 0.25	0.19 0.19	.0002 .0002	.0003 .0003	(14.0) 9.3 (14.0) 9.3
.019	(5.25) 6.0	(5.2) 80.6		16.4	39.3	136.5	25 21	111.5 115.5	0.18 0.19	0.43 0.44	0.34 0.34	.0003 .0003	.0006 .0006	(28.0) 21.0 (28.0) 21.0
.020	(6.75) 7.5	(9.0) 140		16.4	51.0	207.4	50 42	157.4 165.4	0.25 0.27	0.69 0.71 17	0.56 0.57	.0006 .0006	.0012 .0012	(56) 42 (56) 42
.021	(8.25) 9.0	(14.1) 219		16.4	62	297	80 77	217 220	0.35 0.36	1.06 1.07 23	0.88 0.89	.0009 .0009	.0021 .0021	(98) 77 (98) 77
.022	(9.75) 10.5	(20.4) 316		16.4	73	405	120 128	285 277	0.46 0.45	1.53 1.52 35	1.30 1.30	.0013 .0013	.0034 .0034	(158) 128 (158) 128
.023	(11.25) 12.0	(26.7) 415	(ROOF SLAB 15)	16.4	85	516	216 200	300 316	0.49 0.51	2.01 2.03 20	1.77 1.77	.0018 .0018	.0092 .0052	(242) 200 (242) 200
.024	(12.75) 13.5	(32.5) 505		16.4	96	617	290 295	327 322	0.53 0.52	2.50 2.55 24	2.29 2.29	.0023 .0023	.0075 .0075	(348) 295 (348) 295
.025	(14.25) 15.0	(37.8) 582		16.4	107	705	400 414	305 291	0.49 0.47	3.04 3.02	2.80 2.79	.0028 .0028	.0103 .0103	(400) 414 (400) 414
.026	(14.95) 14.9	(40.8) 632		16.4	113	761	520 552	241 209	0.39 0.34	3.41 3.36	3.22 3.19	.0032 .0032	.0135 .0135	(627) 552 (627) 552
				Yield @ ends of girder										
.027	(14.85) 14.8	(42.8) 664		16.4	112	797	670 652	122 140	0.16 0.18	3.52 3.54	3.44 3.45	.0034 .0034	.0169 .0169	(678) 652 (678) 652
.028	(14.75) 14.7	(44.9) 695		16.4	111	922	700 695	122 127	0.16 0.16	3.70 3.70	3.62 3.62	.0036 .0036	.0205 .0205	(712) 695 (712) 695

A.5.5

[redacted]

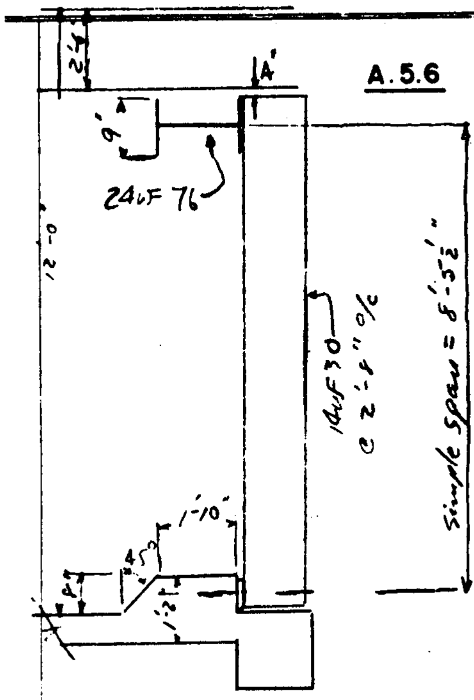
t .028	p	W_r	$\frac{1}{4}wL$	W_s	W_b	W_T	W_R	F	Δv	v_0 3.70	$\bar{v}$	Δx	x .0205	W_r calc (712)695
.029	14.6	(46.8) 725	Roof slab elastic to .032	16.4		851	750 730	101 121	0.13 0.16	3.83 3.86	3.76 3.78	.0038 .0038	.0243 .0243	(747) 730 (747) 730
.030	14.5	(48.2) 746		"		872	760 765	112 107	0.14 0.14	4.00 4.00	3.93 3.93	.0039 .0039	.0282 .0282	(784) 765 (784) 765
.031	14.4	(49.4) 765		"		890	800 803	90 87	0.12 0.11	4.12 4.11	4.06 4.06	.0041 .0041	.0323 .0323	(822) 803 (822) 803
.032	14.3	(49.4) 765		"		889	840	49	0.06	4.17	4.14	.0041	.0364	(858) 840
.033	14.2	(49.4) 765	(-1.9) -29	"		859	858	+1	0	4.17	4.17	.0042	.0406	Yield @ slab
.034	14.1	765	(-1.9) -30	"		858	858	0	0	4.17	4.17	.0042	.0448	
.035	14.0	765	(-2.0) 31	"		956	858	-2	0	4.17	4.17	.0042	.0490	
.040	13.3	765	(-2.0) 34	"		950	858	-8	-.04	4.13	4.15	.0207	.0697	
.0425	13.0 13.1	(49.4) 765	(-2.2) -34	16.4	100	847	858	-11	-.03	4.10	4.12	.0103	.0800	
50	13.0 16.9	(49.4) 765	(-2.4) -37	16.4	98	842	858	-16	-.04	4.06	4.08	.0102	.0902	
75	12.8 12.7	(49.4) 765	(-2.5) -39	16.4	96	838	858	-20	-.05	4.01	4.04	.0101	.1003	
.0500	12.65 12.6	(49.2) 763		16.4	95	874	858	+16	+.04	4.05	4.03	.0101	.1104	
25	12.55 12.5	(46.3) 718		16.4	95	829	858	-29	-.07	3.98	4.01	.0100	.1204	

(max δ reached 0.0715 seconds ; $x = .1623^{FT}$)

A.5.5

[redacted]

A.5.6 Steel Girt - Elastic



wt/' V-beam 2'-8" wide = 8.81 #/'

+ girt = 30

Σ = 38.81 #/'

$\mu = \dfrac{38.81}{12 \times 386} = .00839 \ \dfrac{\# - sec^2}{in} / in$

$L^4 = (101.5)^4 = 1.06 \times 10^8 \ in^4$

$I = 289.6 \ in^4$

$m = \left(\dfrac{38.81 \times 8.458}{32.2}\right)\dfrac{1}{2} = 5.10 \ \dfrac{\# \cdot sec^2}{ft.}$

$T = \dfrac{2\pi}{\omega} = \dfrac{2\pi}{9.87\left[\dfrac{EI}{\mu L^4}\right]^{\frac{1}{2}}}$

$\dfrac{EI}{\mu L^4} = \dfrac{8688 \times 10^6}{.890 \times 10^{-6}} = 9760$

$T = \dfrac{6.28}{9.87 \times 98.8} = 6.44 \times 10^{-3}$ sec

$t_1/T \sim \dfrac{.7}{.644 \times 10^{-2}} = 109$ DCF = 2

Peak pressure = $1.40 \times 16.75 = 23.4$ psi (from Friedrich's equation)

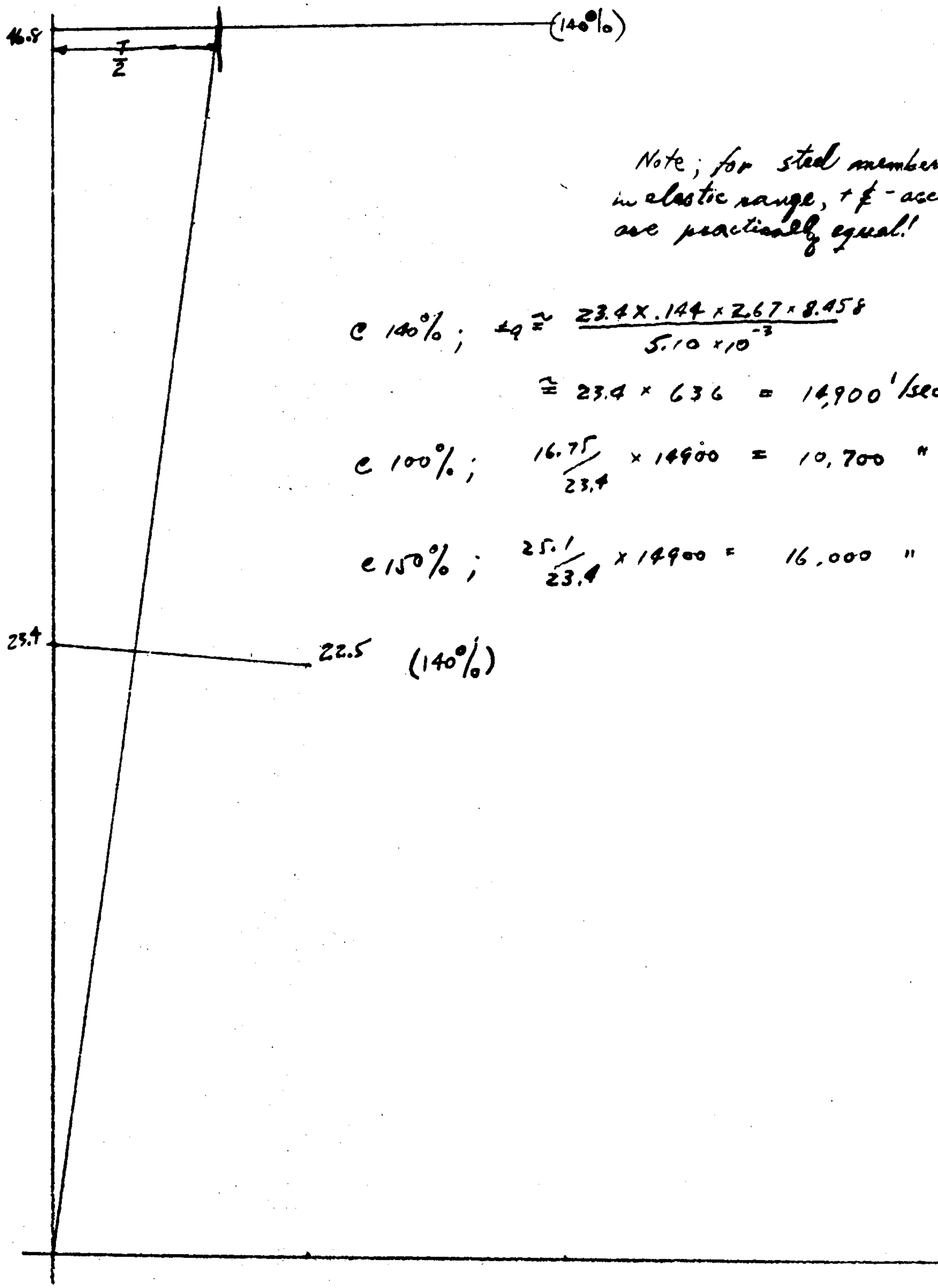
: Elastic design p = 46.8 psi = 6.78 psf

$M_{max} = \dfrac{wl^2}{8} = \dfrac{(6.78 \times 2.67)8.458 \times 101.5}{8} = 1940$ "-k

@ 64 ksi girt good for $64 \times 41.8 = 2680$ "-k ok!

A.5.6

[redacted]



Note; for steel members in elastic range, + & - accel are practically equal!

$$@\ 140\%;\quad a_g \cong \frac{23.4 \times .144 \times 2.67 \times 8.458}{5.10 \times 10^{-3}}$$

$$\cong 23.4 \times 636 = 14{,}900\ '/sec^2$$

$$@\ 100\%;\quad \frac{16.75}{23.4} \times 14900 = 10{,}700\ ''$$

$$@\ 150\%;\quad \frac{25.1}{23.4} \times 14900 = 16{,}000\ ''$$

A.5.6

[redacted]

A.5.7 Steel Girt - Plastic

t	W_B	W_R	W_B-W_R	ΔV	V_0	$\bar{V}$	Δx	x(ft.)	W_R
0					0			0	
	46.35	0.77	45.58	3.26		1.63	.000815		
.0005					3.26			.000815	1.54
	100.17	4.69	95.48	6.84		6.68	.00334		
.0010					10.10			.004155	7.85
	114.33	34.2	80.13	17.2		18.70	.02805		
.0025					27.30			.032205	60.9
	114.00	99.0	15.0	3.19		28.90	.04335		
.0040					30.49			.075555	121.5
	113.82	121.5	-7.68	-0.84		30.07	.02285		
.00476					29.65			.098405	121.5
	103.9	121.5	-17.6	-0.61		29.35	.00705		
.0050					29.04			.105455	121.5
	103.1	121.5	-18.4	-6.58		25.75	.0644		
.0075					22.46			.169855	121.5
	101.4	121.5	-20.1	-7.19		18.86	.0472		
.0100					15.27			.21706	121.5
	99.9	121.5	-21.6	-7.73		11.40	.0285		
.0125					7.54			.24556	121.5
	98.5	121.5	-23.0	-8.23	0	3.77	.00558		
.0150								.25114	121.5

stopped @ t = .01479 sec.

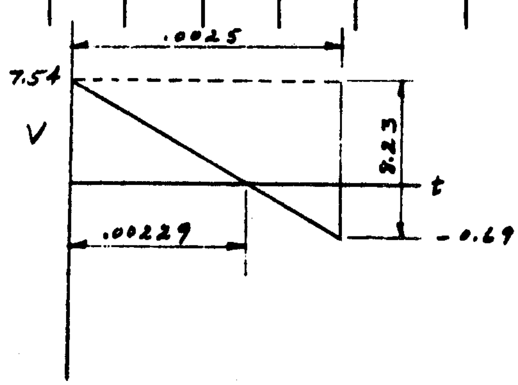
$\frac{\ell}{32} = \frac{12}{32} = .375'$ O.K.

$t = \frac{7.54}{8.23} \times .0025$

$= .00229$

$\underline{.0125}$

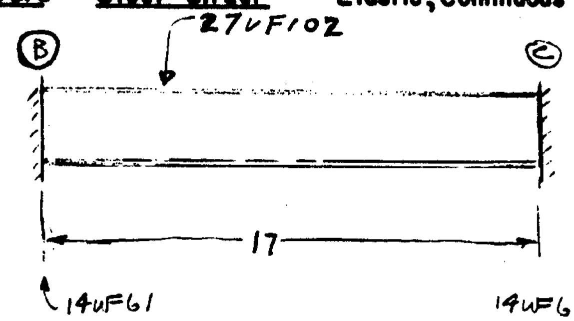
$.01479$

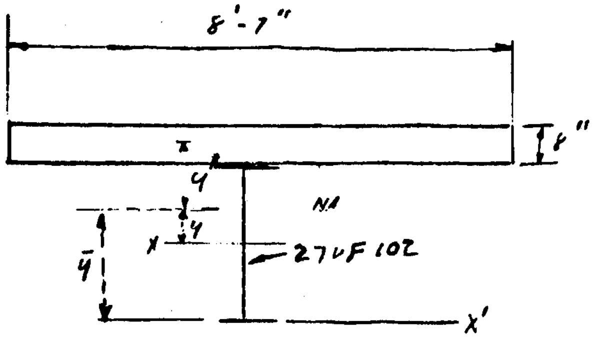


A.5.7

[redacted]

A.5.8 Steel Girder - Elastic, Continuous





$A_c = 82.4$ □"
$A_s = 30.01$

composite beam, n=10

$$Mx': \quad \bar{y} \; \frac{\overset{2560}{(82.4 \times 31.07)} + \overset{406}{(30.01 \times 13.53)}}{\underset{112.41}{82.4 + 30.01}} = 26.4''$$

$$\therefore y = 26.4 - 13.53 = 12.87''$$
$$y' = 31.07 - 26.4 = 4.67''$$

I.

	$\bar{I}$	A	d	d^2	Ad^2	Σ
Conc.	$\frac{10.3 \times 8^3}{12} = 43.8$	82.4	4.67	21.8	1795	1838.8
Steel	3604.1	30.01	12.87	165.0	4960	8564.1
						10402.9 in^4

wt steel = 102 #/l
conc = $\frac{8}{12}$ $\overset{8.58}{\cancel{(5.63)}}$ × 150 = $\cancel{564}$ 560 / $\cancel{686}$ #/l   962 #/l

not required!
$$m = \frac{.686 \times 17}{32.2} = .\cancel{36}2 \; \frac{k - sec^2}{ft.} \quad .508$$
$$eff. \; m = .407 \times .362 = .147 \quad \therefore \frac{1}{m} = \cancel{6.81}$$

A.5.8

[redacted]

$$\omega = 22.4\left[\frac{EI}{M_1 L^4}\right]^{\frac{1}{2}}$$

$$M_1 = \frac{\cancel{666}\ 962 / 12}{386} = .\cancel{148}\ .208 \ \frac{\#\text{-sec}^2}{in} / in$$

$$L^4 = (204)^4 = 17.2 \times 10^8 \ in^4$$

$$M_1 L^4 = \cancel{2.55}\ 3.58 \times 10^8 \qquad \frac{EI}{M_1 L^4} = \cancel{1225}\ 872$$

$$EI = 312087 \times 10^8$$

$$\sqrt{\ } = \cancel{35}\ 29.6$$

$$\omega = \cancel{784}\ 662 \ \text{rdn/sec} \longleftarrow$$

$$f = \frac{\omega}{2\pi} = \frac{784}{628} = \cancel{125}\ 105 \ \text{cps}$$

$$T = \frac{2\pi}{\omega} = \cancel{8.02}\ 9.49 \times 10^{-3} \ \text{sec} \longleftarrow$$

$$t_0/T = \frac{.01}{.00802} = \cancel{1.25}\ 1.05$$

$$t_d/T = \frac{.2}{.008} = \cancel{25}\ 21$$

$$DLF \sim 1.0 \longleftarrow$$

$$\Delta_{max} = \frac{\cancel{1.1}\ 1.0 \times 1.35 \times 2.16 \ k/\square' \times 17 \times 9.54 \times 204^3}{384 \times 30 \times 10^3 \times 10403}$$

$$= \frac{546 \times 9.54 \times 8.23 \times 10^6}{120 \times 9} = \cancel{35.7}\ 32.4 \times 10^{-3} \ '' \longleftarrow ✓$$

$$\ddot{x} \quad \frac{2\pi f x_0}{t_0} = \frac{6.28 \times \cancel{125}\ 105 \times 32.5 \times 10^{-3}}{.01} = \cancel{2550}\ 2140 \ ''/\text{sec}^2 \longleftarrow ✓$$

$$179 \ ''/\text{sec}^2$$

If I is cut by 10% T will increase by ~30%, new DLF ~30% higher & $\ddot{x}$ will be reduced to about 125"/sec², a reduction of about 40%.

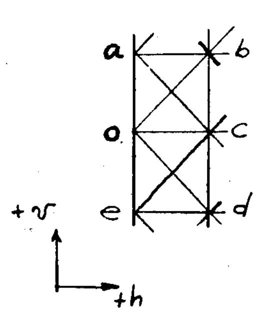
Eff. I may be reduced by cracks or slippage or failure of shear connectors at one end.

A.5.8

[redacted]

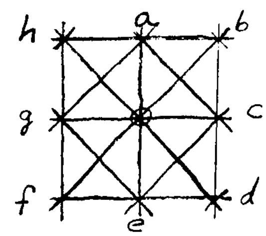
A.5.9 Shear Flow Analysis for Floor Panels

Formulae from Mc Henry's Paper
Lattice Analogy in Concrete Design
Title No. 45-7 A.C.I.



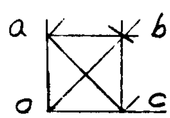
$$h_o = \frac{8}{9E} H_o + \frac{1}{6}(h_b + v_b + 4h_c + h_d - v_d)$$

$$v_o = \frac{8}{9E} V_o + \frac{1}{6}(2v_a + h_b + v_b - h_d + v_d + 2v_e)$$



$$h_o = \frac{4}{9E} H_o + \frac{1}{12}(h_b + v_b + 4h_c + h_d - v_d + h_f + v_f + 4h_g + h_h - v_h)$$

$$v_o = \frac{4}{9E} V_o + \frac{1}{12}(4v_a + h_b + v_b - h_d + v_d + 4v_e + h_f + v_f - h_h + v_h)$$



$$h_o = \frac{2}{3E}(3H_o - V_o) + \frac{1}{4}(-v_a + h_b + v_b + 3h_c)$$

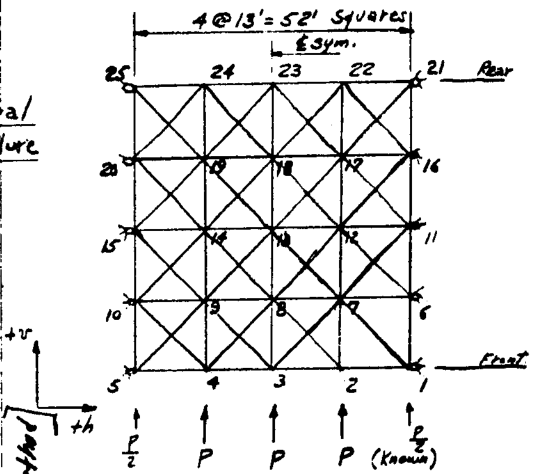
$$v_o = \frac{2}{3E}(-H_o + 3V_o) + \frac{1}{4}(3v_a + h_b + v_b - h_c)$$

Proceed systematically from joint to joint improving successive values until satisfactory convergence is secured.

A.5.9

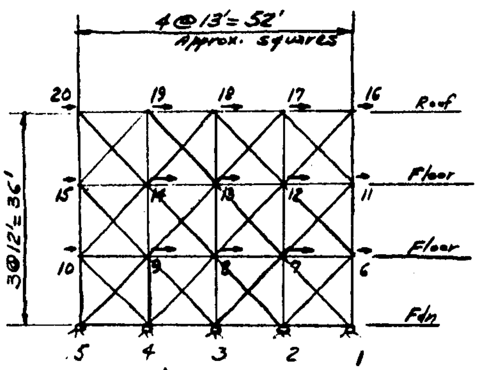
[redacted]

General Procedure



Floor or Roof Slab

Fig. a



Shear Wall

Fig. b

Lattice Analogy - McHenry - Finite Difference Method Title No 45-7 ACI

This 2 dimensional problem assumes superposition applicable & no effect of bending displ. on shear displ. in any one analysis. i.e. Wall remains plane for purposes of wall shear analysis while at the same time the walls have no bending resistance in the normal direction for the analysis of the decks.

* Use displ. terms which account for body loads @ joints. Displ. patterns consistent with loading are automatically prescribed eliminating cut & Try procedure as to displ. Ritter R53

McHenry paper Method
Title No. 45-7 A.C.I.

Boundary Conditions: Assume walls do not permit displ. of deck parallel to walls @ edges. Wall has no bending resistance ⊥ plane of wall. i.e. corner joints have no translation. Assume foundation rigid so that no h or v displ. @ joints 1, 2, 3, 4, 5 in wall.

1. Let $V_3 = 100$ units represent load P @ 3. ($h_3 = 0$). Find the vert load @ 3 corresponding to this 100 unit vert. displ. By direct proportion we can find all displ. due to known vert. force @ 3.

2. Let $V_2 = V_4 = 100$ units. ($h = 0$). Find the vert. loads @ 2, 4 corr. to these 100 unit vert. displ. Compute displ. due to known vert forces @ 2, & 4.

3. Superimpose displ. due to known forces @ 2, 3, 4 & compute strains & stresses.

Note: Steps 1, 2, 3 may be combined by assuming* a displ. pattern @ joints 2, 3, 4 & finding the boundary loading due to this displ. pattern. This loading pattern must be similar in shape to known loading so that we can compute displ. due to actual loading by direct proportion. Compute strains & stresses. This a cut & try process.

4. Apply the deck reactions as the loading on the wall. Displ. patterns must be assumed* @ each deck & adjusted accordingly. displ. analyses may be superimposed & final stresses computed in walls & decks.

Apply 100 unit vert. displ. @ joints ② & ④ in Fig. a.

Relaxation Schedule (R.S. 2)

h 1	v	h 2	v	h 3	v	h 6	v	h 7	v	h 8	v	h 11	v	h 12	v	h 13	v	h 16	v	h 17	v	h 18	v	h 21	v	h 22	v	h 23	
0	0	0	100	0	150	15	0	15	70	0	130	13	0	10	50	0	100	10	0	8	40	0	60	0	0	0	20	0	40
0	0	0	100	0	50	20	0	4	64	0	77	13	0	8	45	0	64	11	0	7	29	0	44	0	0	0	25	0	19
0	0	0	100	0	74	13	0	6	61	0	72	13	0	9	40	0	54	9	0	7	27	0	34	0	0	0	22	0	29
0	0	0	100	0	70	16	0	10	58	0	66	14	0	9	36	0	47	9	0	6	24	0	28	0	0	0	19	0	25
0	0	0	100	0	67	19	0	14	54	0	59	16	0	10	30	0	40	9	0	5	20	0	22	0	0	0	16	0	20
0	0	0	100	0	62	23	0	20	50	0	57	16	0	11	29	0	36	7	0	5	18	0	24	0	0	0	15	0	20
0	0	0	100	0	57	29	0	26	44	0	55	18	0	13	27	0	31	5	0	4	15	0	24	0	0	0	13	0	19
0	0	0	100	0	60	32	0	15	55	0	54	19	0	12	28	0	35	7	0	5	17	0	23	0	0	0	14	0	19
0	0	0	100	0	59	29	0	13	52	0	55	17	0	11	28	0	36	8	0	6	17	0	24	0	0	0	14	0	20
0	0	0	100	0	58	23	0	11	52	0	55	16	0	11	28	0	37	8	0	5	17	0	24	0	0	0	14	0	20
0	0	0	100	0	58	21	0	10	52	0	55	16	0	10	29	0	37	8	0	5	18	0	25	0	0	0	15	0	21
0	0	0	100	0	57	20	0	10	52	0	55	15	0	10	28	0	38	7	0	5	18	0	25	0	0	0	15	0	21
0	0	0	100	0	57	20	0	10	52	0	55	15	0	10	29	0	38	9	0	6	18	0	25	0	0	0	15	0	21
0	0	0	100	0	57	20	0	10	52	0	55	15	0	10	29	0	38	8	0	5	18	0	25	0	0	0	15	0	21
0	0	0	100	0	57	20	0	10	52	0	55	15	0	10	29	0	38	7	0	5	18	0	25	0	0	0	15	0	21
0	0	0	100	0	57	20	0	10	52	0	55	15	0	10	29	0	38	7	0	5	18	0	25	0	0	0	15	0	21

A.5.9

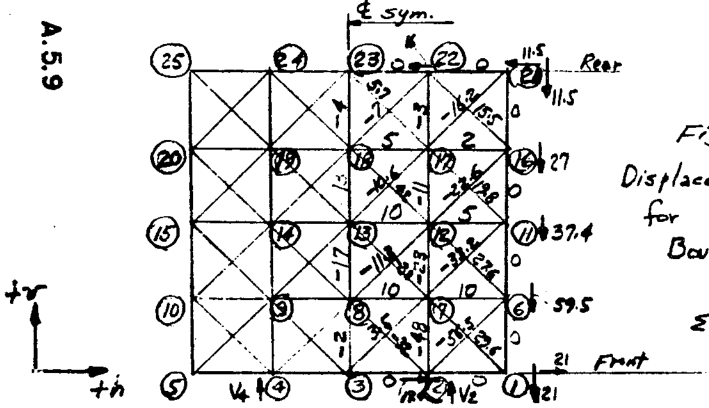


Fig. d

Displacement (e) Diagram
for 100 unit vert. displ. @ ②&④.
Bound. Cond. as per procedure (p.4).

$\Sigma V = 0$; $\Sigma H = 0$

$V_2 = V_4 = 159$ causing 100 unit displ. @ ②&④

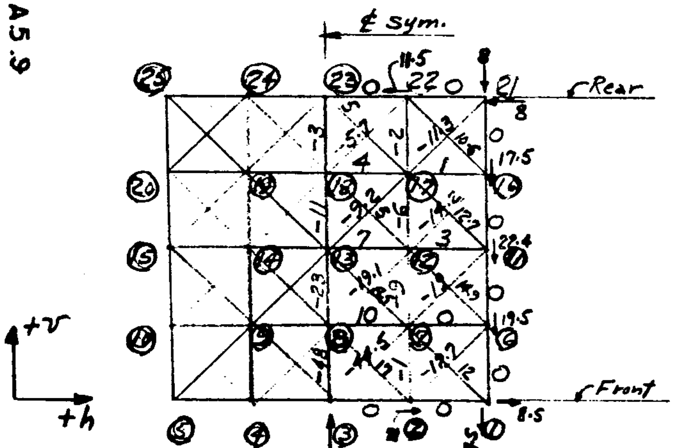
[redacted]

Apply 100 unit vert. displ. @ joint 3 in fig a.

start off with approx. as to displ.

Relaxation Schedule (R.S. 1)

h 1	v	h 2	v	h 3	v	h 6	v	h 7	v	h 8	v	h 11	v	h 12	v	h 13	v	h 16	v	h 17	v	h 18	v	h 21	v	h 22	v	h 23	
0	0	0	30	0	100	15	0	10	15	0	50	25	0	15	10	0	35	15	0	10	7	0	20	0	0	0	4	0	10
0	0	0	21	0	100	13	0	12	24	0	53	17	0	11	17	0	29	11	0	7	9	0	15	0	0	0	7	0	11
0	0	0	27	0	100	12	0	11	25	0	52	13	0	9	17	0	27	8	0	5	10	0	15	0	0	0	8	0	12
0	0	0	27	0	100	11	0	11	26	0	51	11	0	8	17	0	27	6	0	4	11	0	16	0	0	0	9	0	13
0	0	0	28	0	100	11	0	11	27	0	52	11	0	8	18	0	28	6	0	4	12	0	17	0	0	0	10	0	15
0	0	0	29	0	100	11	0	11	27	0	52	10	0	8	18	0	28	5	0	4	12	0	18	0	0	0	10	0	15
0	0	0	29	0	100	11	0	11	27	0	52	10	0	8	18	0	29	5	0	4	12	0	18	0	0	0	10	0	15
0	0	0	29	0	100	11	0	10	27	0	52	10	0	8	18	0	29	5	0	4	12	0	18	0	0	0	10	0	15
0	0	0	29	0	100	10	0	10	27	0	52	10	0	7	18	0	29	5	0	4	12	0	18	0	0	0	10	0	15
0	0	0	28	0	100	10	0	10	27	0	52	10	0	7	18	0	29	5	0	4	12	0	18	0	0	0	10	0	15
0	0	0	28	0	100	10	0	10	27	0	52	10	0	7	18	0	29	5	0	4	12	0	18	0	0	0	10	0	15



$V_3 = 159$ causes 100 unit vert. displ. @ 3

Fig. c Displacement (e) Diagram
for 100 unit vert. displ. @ 3
Bound. Cond. as per. procedure (p.4).

$\Sigma V = 0$
$\Sigma H = 0$

Deck Analysis

Use figs. c & d & superimpose joint @ 1 for 2nd. Fl. $\underline{1320^K}$ @ deck joints 2, 3, 4 (6" slab)

$$V_1 = \frac{1320}{159}(21+8.5) = 245^K \quad + \underbrace{660}_{\text{Local Blast}} = \underline{905^K}$$

$$V_6 = \frac{1320}{159}(59.5+19.5) = \underline{655^K}$$

$$V_{11} = \frac{1320}{159}(32.4+22.4) = \underline{496^K}$$

$$V_{16} = \frac{1320}{159}(27+17.5) = \underline{370^K}$$

$$V_{21} = \frac{1320}{159}(11.5+8) = \underline{162^K}$$

→ shear of 2nd Fl. on reaction shear wall

displ. @ 3 = (100+57) × .001230 = .193" (max. for deck) See p. 4

Some load goes directly into shear wall from front wall

$$\frac{1320}{2} = 660^K$$

$$3^{rd}\ Fl\ \text{shears} = \frac{970}{1320} \times 2^{nd}\ Fl.$$

$$\text{Roof shears} = \frac{269}{1320} \times 2^{nd}\ Fl.$$

1^{st} Fl. no floor slab $\quad 53.1^K/ft \times 9.2' = \underline{489^K}$ resisted at A (rear) see fig. K p. 10

A.5.9

[redacted]

Stresses in panel 11-6-7-12

Refer pp. 546 for displ. values

$\epsilon_h = \frac{13+19}{2} \times \frac{.000652}{13\times12} = 67\times10^{-6}$ in/in $\qquad \frac{.000652}{2\times13\times12} = 2.09\times10^{-6}$

$\epsilon_v = -59\times 2.09\times10^{-6} = -123\times10^{-6}$

Critical Max. Tensile principal stress

$\epsilon_{11-7} = -2.09\times10^{-6}\times100\sqrt{2} = -296\times10^{-6}$

$\epsilon_{6-12} = 2.09\times10^{-6}\times81.4\sqrt{2} = +240\times10^{-6}$

$\sigma_h = \frac{36}{35} E\left(\epsilon_h + \frac{1}{6}\epsilon_v\right) = \frac{36}{35}\times3\times10^{6}\left(67-\frac{1}{6}\times123\right)\times10^{-6} = +144$ #/□"

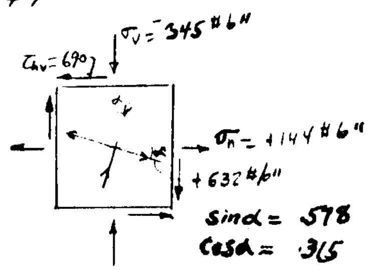
$\sigma_v = \frac{36}{35}\times3\times10^{6}\left(-123+\frac{1}{6}\times67\right)\times10^{-6} = -345$ #/□"

$\gamma = \epsilon_{11-7} - \epsilon_{6-12} = (-296-240)\times10^{-6} = -536\times10^{-6}$

$\tau_{hv} = G\gamma = \frac{3}{7}\times3\times10^{6}\times(-536)\times10^{-6} = -690$ #/□"

$\tau_{max} = \sqrt{\left(\frac{\sigma_h-\sigma_v}{2}\right)^2+\tau_{hv}^2} = \sqrt{\left(\frac{144+345}{2}\right)^2+690^2} = 732$ #/□"

$\sigma_{I,II} = \frac{\sigma_h+\sigma_v}{2} \pm \tau_{max} = \frac{144-345}{2} \pm 732 = \begin{matrix}+632\\-832\end{matrix}$ #/□"



$\sin\alpha = .578$
$\cos\alpha = .815$

$\tan 2\alpha = \frac{\tau_{hv}}{\frac{1}{2}(\sigma_h-\sigma_v)} = \frac{690}{\frac{1}{2}(144+345)}$

$\tan 2\alpha = 2.83$
$2\alpha = 70.6°$
$\alpha = +35.3°$

Stresses in panel 12-7-8-13

not critical

$\epsilon_h = \frac{31+36}{2} \times \frac{.000652}{13\times12} = +140\times10^{-6}$ in/in

$\epsilon_v = \frac{-59-75}{2} \times \frac{.000652}{13\times12} = -280\times10^{-6}$

$\epsilon_{12-8} = -2.09\times10^{-6}(56.6)\sqrt{2} = -167\times10^{-6}$

$\epsilon_{7-13} = 2.09\times10^{-6}\times 9.2\sqrt{2} = +27.2\times10^{-6}$

$\sigma_h = \frac{36}{35}\times3\times10^{6}\times\left(140-\frac{1}{6}\times280\right)\times10^{-6} = +288$ #/□"

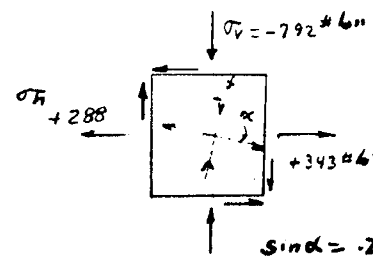
$\sigma_v = \frac{36}{35}\times3\times10^{6}\left(-280+\frac{1}{6}\times140\right)\times10^{-6} = -792$ #/□"

$\gamma = \epsilon_{12-8} - \epsilon_{7-13} = (-167-27.2)\times10^{-6} = -194\times10^{-6}$

$\tau_{hv} = G\gamma = \frac{3}{7}\times3\times10^{6}\times194\times10^{-6} = 250$ #/□"

$\tau_{max} = \sqrt{\left(\frac{\sigma_h-\sigma_v}{2}\right)^2+\tau_{hv}^2} = \sqrt{\left(\frac{288+792}{2}\right)^2+250^2} = 595$ #/□"

$\sigma_{I,II} = \frac{\sigma_h+\sigma_v}{2} \pm \tau_{max} = \frac{288-792}{2} \pm 595 = \begin{matrix}+343\\-847\end{matrix}$ #/□"



$\sin\alpha = .216$
$\cos\alpha = .976$

$\tan 2\alpha = \frac{\tau_{hv}}{\frac{1}{2}(\sigma_h-\sigma_v)} = \frac{250}{\frac{1}{2}(288+792)}$

$\tan 2\alpha = .463$
$2\alpha = 25°$
$\alpha = 12.5°$

A.5.9

[redacted]

A.5.10 Stability Analysis

		TRANS. FORCES					DECK FORCES				BLAST WALL MOMENTS			
t	tau	F-A	2	B	3	C-R	F	2	3	R	F×4'	2×16'	3×28'	R×40'
0														
.005	.0025	47.5	46.8	46.0	45.8	45.5	1090	2143	2100	1040	4360	34400	58800	40,800
.010	.0075	45.0	44.5	44.0	42.0	40.0	1030	2040	1925	917	4120	32700	54000	36700
.015	.0125	43.0	42.8	42.5	37.3	36.0	985	1960	1800	825	3940	31400	50500	33000
.020	.0175	41.5	41.0	40.5	37.0	33.5	951	1880	1615	768	3810	30100	47500	30800
.030	.025	39.5	37.8	36.0	33.3	30.5	905	1730	1525	700	3620	27700	42700	28000
.040	.035	32.5	32.0	31.5	29.3	27.0	745	1470	1340	619	2980	23500	37600	24800
.050	.045	27.5	27.5	27.5	25.5	23.5	630	1260	1170	539	2520	20200	32800	21600
.060	.055	24.5	24.3	24.0	22.5	21.0	562	1115	1030	481	2245	17850	28900	19300
.070	.065	21.0	20.8	20.5	19.8	19.0	481	953	908	435	1925	15250	25420	17400
.080	.075	18.0	18.0	18.0	17.8	17.5	413	825	815	401	1650	13200	22800	16100
.090	.085	15.5	15.8	16.0	16.0	16.0	355	724	735	366	1420	11600	20600	14650
.100	.095	13.5	13.6	13.6	14.0	14.5	309	623	642	332	1235	9970	17950	13300
.150	.125	9.5	9.8	10.0	10.3	10.5	218	450	473	240	873	7200	13250	9600
.200	.175	6.0	6.3	6.5	6.3	6.0	138	289	289	138	552	4630	8100	5520
.250	.225	4.0	4.0	4.0	3.8	3.6	92	184	174	83	368	2950	4870	3320

DECK FORCE = pressure (p.s.i.) × $\frac{144}{1000}$ × 12' × 26.5' = 45.8×p kips (floors)
22.9×p kips (roof & fnd'n)

DL + LL of 100#/□ on floors — 8.5' perimeter only.

		P(kips) × l	=	M_{DL+LL} ('k)
12" SHEAR WALL	52 × 36 × .15 = 280K	× 25.5	=	7150
12" FRONT "	25.5 × 36 × .15 = 138	× 51	=	7050
11" ROOF SLAB	50 × 8.5 × .92 × .15 = 58.6	× 25.5	=	1500
	2 × 18 × 8.5 × .92 × .15 = 42.2	× 25.5	=	1080
6" FLOOR "	2 × 50 × 8.5 (.075 + .1) = 148.5	× 25.5	=	3790
	2 × 2 × 18 × 8.5 (.075 + .1) = 107	× 25.5	=	2730
FRONT WALL FTG.	4 × 4 × 26.5 × .150 = 63.5	× 51	=	3240
SHEAR WALL FTG.	2.5 × 4 × 47 × .150 = 70.5	× 25.5	=	1800
STRAP FTG. (part)	2 × 4 × 4 × 6.5 × .150 = 31.2	× 25.5	=	800
				29140 1K

with 135#/cuft concrete:

	252K	× 25.5	=	6430
	124	× 51	=	6320
	53	× 25.5	=	1350
	38	× 25.5	=	970
850 (.168) =	143	× 25.5	=	3650
72 × 8.5 (.168) =	103	× 25.5	=	2630
	57	× 51	=	2910
	63	× 25.5	=	1600
	28	× 25.5	=	715
				26575

A.5.10

[redacted]

t	Pressures (psi) $t_{avl.}$	p_{12}	p_{23}	p_{34}	Roof Forces F_{12}	F_{23}	F_{34}	M_{12} $F\times 47.2$	M_{23} $F\times 25.5$	M_{34} $F\times 3.75$	ΣM
0	.0025	0	0	0	0	0	0	0	0	0	0
.005	.0075	16.7	0	0	541	0	0	25600	0	0	25600
.010	.0125	17.0	0	0	550	0	0	26000	0	0	26000
.015	.0175	16.5	4.0	0	535	171	0	25220	4360	0	29580
.020	.025	14.8	15.0	1.0	480	640	32	22700	16300	120	39120
.030	.035	11.5	13.8	6.5	372	590	210	17600	15150	790	33540
.040	.045	8.0	12.8	12.5	260	546	405	12300	13950	1520	27770
.050	.055	6.0	12.5	11.8	194	535	362	9170	13650	1360	24180
.060	.065	3.5	12.0	10.5	114	513	340	5400	13100	1280	19780
.070	.075	2.0	11.8	10.0	65	505	324	3070	12900	1220	17190
.080	.085	1.5	11.3	9.0	49	483	292	2320	12350	1095	15770
.090	.095	1.4	10.8	8.2	46	461	266	2180	11760	1000	14940
.100	.125	1.2	8.5	6.5	39	364	210	1840	9300	790	11930
.150	.175	1.0	2.5	4.5	32	107	146	1510	8150	547	10210
.200	.225	0.8	0.7	3.2	26	30	104	1230	7650	390	9270
.250											

Roof Force $F_{23} = p \times 35 \times 8.5 \times \frac{144}{1000}$
$= 427\ p$

Roof Force F_{12} or $F_{34} =$
$p \times 26.5 \times 8.5 \times \frac{144}{1000} = 324\ p^{*}$

t	Footing t_{avg}	p	M
0	.0025	47.5	38,700
.005	.0075	45.0	36700
.010	.0125	43.0	35100
.015	.0175	41.5	33900
.020	.025	39.5	32200
.030	.035	32.5	26500
.040	.045	27.5	22500
.050	.055	24.5	20000
.060	.065	21.0	17100
.070	.075	18.0	14700
.080	.085	15.5	12650
.090	.095	13.5	11000
.100	.125	9.5	7750
.150	.175	6.0	4900
.200	.225	4.0	3260
.250			

Footing Force = pressure$(p) \times \frac{144}{1000} \times 4' \times 26.5 = p \times 1525$ kips
$M = p \times 15.25 \times 53.5' = 815\ p$ (ft kips) A.5.10

[redacted]

A.5.10

t	Δt	Blast Moments Wall	Blast Moments Roof	Blast Moments FTG	D+L Mom.	ΣM	$\Delta\omega = \frac{\Sigma M \Delta t}{I}$	ω	ϖ	Δ29	29	29×51.5 Δ ft.
100% BLAST												
0	.005	138400	0	-38700	(26580) -29140	70600	+.00690	0	.00335	.0000168	0	
.005	.005	127500	-25600	-36700		36100	+.00343	.00670	.00841	.0000421	.0000168	
.010	.005	118840	-26000	-35100		28600	+.00272	.01013	.01149	.0000574	.0000589	
.015	.005	112200	-29580	-33900		19580	+.00186	.01285	.01300	.0000650	.0001163	
.020	.010	102000	-39120	-32200		1540	+.00029	.01314	.01311	.0001311	.0001813	
.030	.010	88900	-33540	-26500		-280	-.00005	.01309	.01287	.0001287	.0003124	
.040	.010	77100	-27770	-22500		-2310	-.00044	.01265	.01208	.0001208	.0004411	
.050	.010	67300	-24180	-20000		-6000	-.00114	.01151	.01094	.0001094	.0005619	
.060	.010	60000	-19780	-17100		-6000	-.00114	.01037	.00968	.0000968	.0006713	
.070	.010	53150	-17190	-14700		-7280	-.00139	.00898	.00810	.0000810	.0007681	
.080	.010	48270	-15770	-12650		-9290	-.00176	.00722	.00602		.0008491	.0437 ft max
.090	.010	42460	-14940	-11000		-12620	-.00240	.00482				
.100	.050	30920	-11930	-7750		-17900	-.01700					
.150	.050	18800	-10210	-4900		-25450	-.02450					
.200												
125% BLAST												
0	.005	173000	0	-48400	(26580) -29140	(97960) 95460	(.01010) +.00905		(.00505) .00453	(.0000252) .0000227		
.005	.005	159500	-32000	-45900		(55000) 52460	(.00578) +.00498	(.01810) .00905	(.01295) .01154	(.0000648) .0000577		
.010	.005	148400	-32500	-44900		(44360) 41860	(.00460) +.00398	(.01580) .01403	(.01810) .01602	(.0000905) .0000802		
.015	.005	140500	-36900	-42400		(34500) 32060	(.00357) +.00304	(.02040) .01801	(.02219) .01953	(.0001110) .0000978		
.020	.010	127500	-48900	-40300		(11660) 9160	(.00240) +.00174	(.02397) .02105	(.02517) .02192	(.0002517) .0002192		
.030	.010	111000	-41900	-33100		(9360) 6860	(.00194) +.00138	(.02637) .02279	(.02734) .02322	(.0002734) .0002322		
.040	.010	96500	-34600	-28100		(7160) 4660	(.00148) +.00085	(.02831) .02364	(.02905) .02363	(.0002905) .0002363		
.050	.010	84200	-30200	-25000		(2400) -140	+(.00050) -.00003	(.02979) .02361	(.03004) .02359	(.0003004) .0002359		
.060	.010	75000	-24700	-21400		(2000) -240	+(.00040) -.00004	(.03029) .02357	(.03049) .02340	(.0003049) .0002340		
.070	.010	67200	-21500	-18400		(700) -1840	+(.00001) -.00035	(.03069) .02322	(.03070) .02282	(.0003070) .0002282		
.080	.010	60400	-19700	-15800		(-1500) -4240	(-.00030) -.00081	(.03070) .02241	(.03060) .02146	(.0003060) .0002146		
.090	.010	53000	-18680	-13700		(6000) -8520	-(.00175) -.00190	(.03049) .02051	(.02997) .01332	(.0002997) .0001332		
.100	.050	38600	-14900	-9900		(-12600) -15140	(-.01300) -.01498	(.02924) .00613	(.02274)	(.0002274)		(.1470') .1026 ft MAX.
.150	.050	23500	-12800	-6100		(-22000) -24540	(-.02300) -.02420	(-.01624) —			.0019920 (.0028525)	

NOTE: FIGS IN ( ) ARE FOR 135#/cu.ft. CONCRETE

[redacted]

A.5.11 Shear Reinforcement for Walls

SYSTEM No. 4 (USING DATA FROM BULL. #175, U. OF ILLINOIS ENG. EXP. STATION)

FOR VERTICAL STIRRUPS – $r = \frac{1.33V - 0.250}{50}$

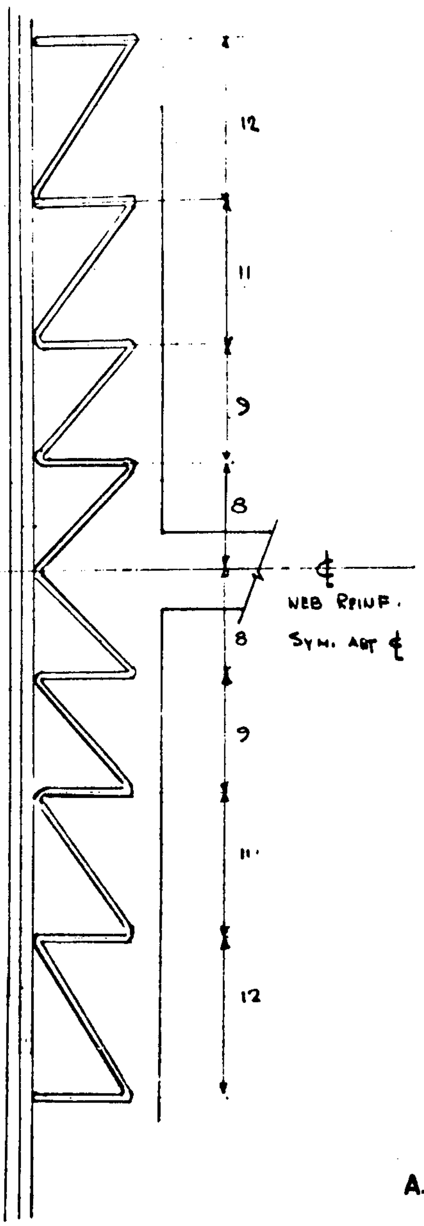
INCLINED STIRRUPS $r = \frac{1.33\frac{V}{K} - .250}{50}$ $\quad K = (\sin\alpha + \cos\alpha)\sin\alpha$

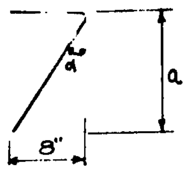
$v_{MAX} = 0.478\ ^{K}/_{in^2}$

$S = 3.12^{FT}$

MAX $r_{VERT\ STIRR} = \frac{1.33 \times .478 - .250}{50} = 0.78\%$ $\quad A_w = 6.5 \times .0078 = .050\ ^{in^2}/_{in}$ FOR $6\frac{1}{2}$" WIDTH

MAX $r_{INCL} = \frac{1.33 \times .478}{50K} - \frac{.250}{50} = \frac{.0127}{K} - 0.0050$ $\quad A_w = \left(\frac{.083}{K} - .0022\right)^{in^2}/_{in}$ FOR $6\frac{1}{2}$"





WITH – $\frac{1}{2}\phi$ BARS $(A_s = .20)$

LET $a_1 = 7$"

$\tan\alpha = 8/7 \quad \alpha = 48.8°$

$K = (.753 + .656) .753 = 1.06$ (1.403)

$S_{VER.ST} = \frac{.20}{.050} = 4.00$

$S_{INC.ST} = \frac{.20}{.078 - .032} = 4.35$

8.35"

TRY $a_1 = 8 \quad K = 1 \quad S = 4.0$

$+ 4.0 = 8.0$" OK

$r_{2\,V.S.} = \frac{32.5}{37.5}\left[\frac{1.33 \times .478 - .250}{50}\right] = .070$ $\quad A_s = .045\ ^{in^2}/_{in}$

$r_{2\,I.S.} = \frac{.011}{K} - .004$ $\quad A_s = \left(\frac{.071}{K} - .028\right)^{in^2}/_{in}$

LET $a_2 = 9$" $\quad \alpha = \tan^{-1}\frac{8}{9} = 41.6°$

$K = (.665 + .748).665 = .942$ (1.413)

$S = \frac{.2}{.043} \quad 4.65$

$+ \frac{.2}{.047} = 4.30 \quad 8.95$ OK

$r_{3\,VS} = .0045 \quad A_s = .032\ ^{in^2}/_{in}$

$r_{3\,I.S} = \frac{.008}{K} - .0031 \quad A_s \left(\frac{.052}{K} - .020\right)^{in^2}/_{in}$

LET $a_3 = 11 \quad \alpha = \tan^{-1}\frac{8}{11} = 36° \quad K = [.588 + .810].588 = .824$

$S_{VS} = .20/.032 = 6.2$

$S_{IS} = \frac{.20}{.063 - .020} = 4.9 = 11.1$" OK

USE SYSTEM #4

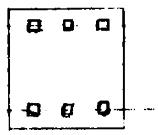
A.5.11

[redacted]

3-4 - INT.

Concrete Frame with Windows

A.5.12 Columns -- P - M & δ_e



Top & Bottom

$6 - 1\frac{1}{8}'' \square \rightarrow 7.59 \square''$

16 x 16

$p_c = 256$

$p_t = .0296$

$d' = 11.875$

$d = 13.94$, $d^2 = 194$

TENSION; Top & Bottom -

$$P = \frac{18.3 \times 10^4}{P} + 653 - 81.6\,e$$

P	$\frac{18.3 \times 10^4}{P}$	+653	P -	÷ 81.6 = e	M
50	3660	4313	-4263	52.2	218 ✓
100	1830	2483	-2383	29.2	244 ✓
200	915	1568	-1368	16.75	279 ✓
300	610	1263	-963	11.8	295 ✓
400	457.5	1110.5	-710.5	8.71	290 ✓

COMPRESSION; Top & Bottom

$$P = \frac{379}{.168e + 1} + \frac{768}{.247e + 1.178}$$

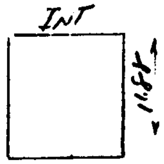
e		P	M
0	379 + 652	1031	0
10	141 + 210	351	292

A.5.12

[redacted]

3-4

INT



Top & Bottom

16×16

$6-1\frac{1}{8}^{\square} \rightarrow 7.59$

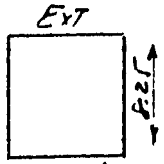
$I_c = \qquad 5460$

$10 I_s = 10(7.59 \times 5.94^2) = \underline{2670}$

8130 in^4

$= \underline{.392 \text{ ft}^4}$

EXT



Top & Bottom

12×12

$6-\frac{3}{4}^{\phi} \rightarrow 2.65$

$I_c = \frac{12^3}{=} \qquad 1728$

$10 I_s = 10(2.65 \times 4.13^2) = \underline{451}$

2179

$= .105 \text{ ft}^4$

$$M = \frac{6EI\Delta}{L^2} = 2.592 \times 10^6 \frac{I\Delta}{L^2}$$

$L_{1-2} = 9.5'$

$L_{2-3} = 9.7'$

$L_{3-4} = 9.5'$

1-2	EXT	$M = 2.592 \times 10^6 \times .832 \div 90$	=	$24.0 \times 10^3\ \Delta$
	INT	" 2.07 "	=	59.6 "
2-3	E	$M =$ " $.388 \div 94$	=	10.7 "
	I	" 1.26 "	=	34.7 "
3-4	E	" $.105 \div 90$	=	3.02 "
	I	" $.392$ "	=	11.3 "

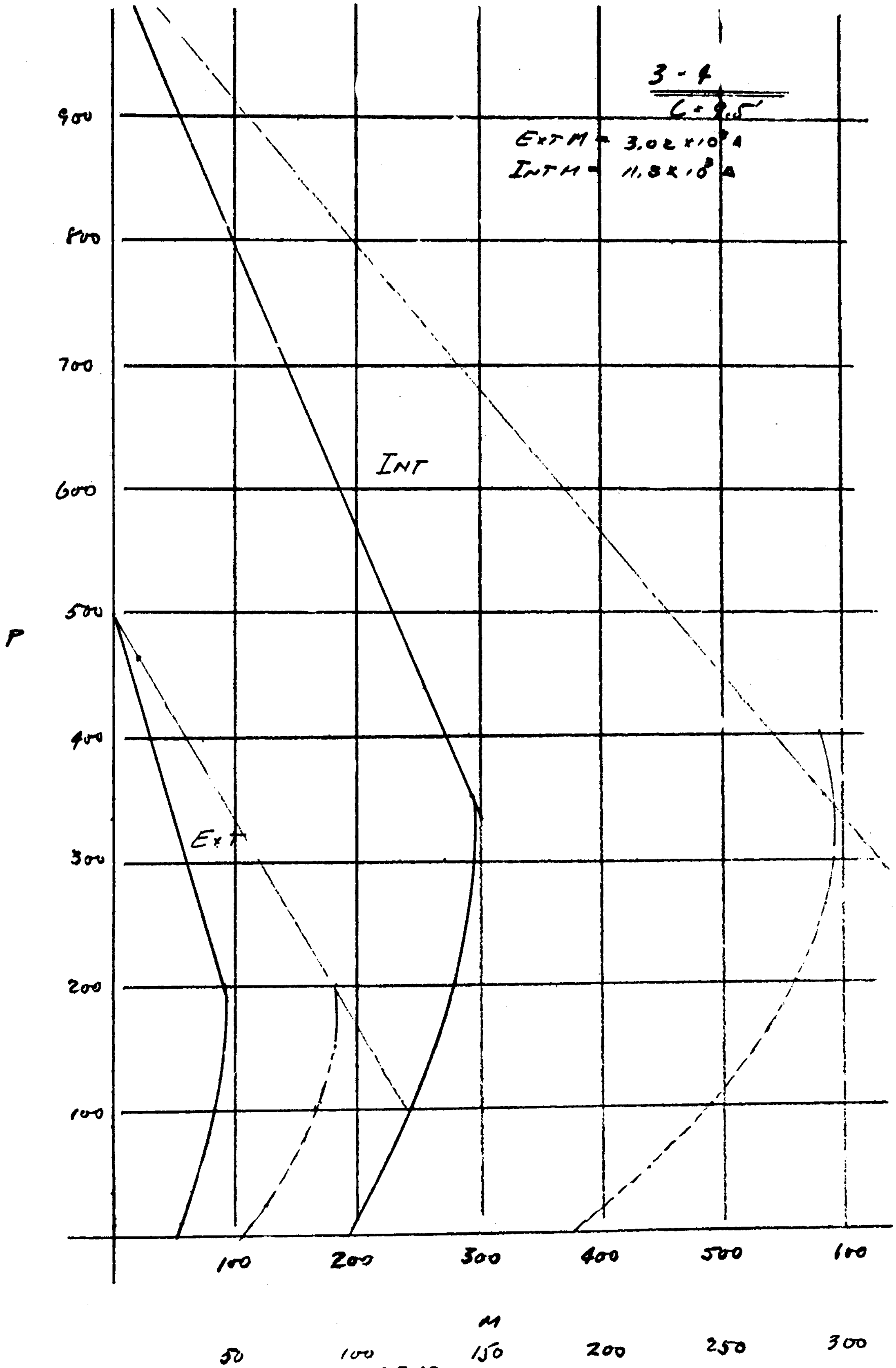
WALLS :

$I_c = .0834 \text{ ft}^4$ (neglect steel)

$$\therefore FM_\Delta = \frac{6EI\Delta}{L^2} = \left[\frac{2.592 \times 10^6 \times .0834}{144}\right]\Delta = 1500\Delta$$

A.5.12

[redacted]



A.5.12

[redacted]

A.5.13 Loads

	3-4							
	A 25		B 38		C 38		D 25	
	P	M	P	M	P	M	P	M
0								
.01	140	90	153	264	38	212	25	61
.02	143	91	191	274	73	230	25	61
.03	87	79	209	278	171	269	49	68
.04	61	72	180	271	220	282	101	82
.05	42	66	154	264	222	282	110	84
.075	13	56	110	249	192	274	95	80
.10	1	51	73	230	153	264	81	72
.125	-5	51	40	213	102	247	57	70
.150	-5	51	13	200	61	224	43	66
.175	-3	51	-10	193	11	199	18	57
.200	1	51	-18	193	5	196	24	61
.225	5	53	-18	193	-4	193	19	58
.250	7	54	-12	193	-2	193	17	57

	2-3							
	A 45		B 69		C 69		D 45	
	P	M	P	M	P	M	P	M
0								
.01	160	190 230	184	573 668	69	501 596	45	141 182
.02	163	191 231	222	596 691	104	523 620	45	141 182
.03	107	169 209	240	605 703	202	584 678	69	152 192
.04	81	156 196	211	589 685	251	610 710	121	174 214
.05	62	149 189	185	573 668	253	612 712	130	178 218
.075	33	131 172	141	548 643	223	597 692	115	170 210
.10	21	124 155	104	523 620	184	572 667	101	163 203
.125	15	132 174	71	501 596	133	544 539	77	154 194
.150	15	132 174	44	482 577	92	515 610	63	149 189
.175	17	133 175	21	470 565	42	480 575	38	138 178
.200	21	124 155	13	456 551	36	475 570	44	141 182
.225	25	127 168	13	456 551	27	467 562	39	140 179
.250	27	128 169	19	469 564	29	470 564	37	137 177

	1-2							
	A 67		B 103		C 103		D 67	
	P	M	P	M	P	M	P	M
0								
.01	182	326	218	828	103	738	67	254
.02	185	328	256	854	138	768	67	254
.03	129	296	274	865	236	823	91	269
.04	103	278	245	846	285	871	143	305
.05	84	265	219	828	287	873	152	311
.075	55	244	175	797	257	855	137	300
.10	43	237	138	768	218	828	123	292
.125	37	233	105	740	167	792	99	275
.150	37	233	78	713	126	757	85	266
.175	39	235	55	695	76	712	60	250
.200	43	237	47	688	70	708	66	254
.225	47	239	47	688	61	700	61	251
.250	49	241	53	693	63	701	59	249

A.5.13'

[redacted]

Wall Loads for 1'-0" Strip   p @ 100% of blast

$$W_p = p \times \frac{12 \times 144}{1000} = 1.728\, p$$

t	Δt	Front 2nd Floor W_p	Front 2nd Floor ΔW_p	Front 3rd Floor W_p	Front 3rd Floor ΔW_p	Front 4th Floor W_p	Front 4th Floor ΔW_p	Rear 2nd Floor W_p	Rear 2nd Floor ΔW_p	Rear 3rd Floor W_p	Rear 3rd Floor ΔW_p	Rear 4th Floor W_p	Rear 4th Floor ΔW_p
0		80.5		80.5		80.5		0		0		0	
	.01		-15.6		-19.0		-26		0		0		0
.01		64.9		61.5		54.5		0		0		0	
	.01		-6.1		-7.7		-8.9		0		0		0
.02		58.8		53.8		45.6		0		0		0	
	.01		-12.3		-7.3		-6.8		0		0		0
.03		46.5		46.5		38.8		0		0		0	
	.01		-7.4		-6.9		-4.4		+22.6		+21.5		+10.3
.04		39.1		39.6		34.4		22.6		21.5		10.3	
	.01		-3.7		-4.6		-4.1		-15.2		-14.6		+0.6
.05		35.4		35.0		30.3		7.4		6.9		10.9	
	.025		-8.8		-9.0		-7.4		-6.9		-4.7		+0.3
.075		26.6		26.0		22.9		0.5		2.2		11.2	
	.025		-5.8		-6.0		-5.7		-2.7		-2.7		-6.4
.100		20.8		20.0		17.2		-2.2		-0.5		4.8	
	.025		-4.3		-3.9		-3.1		0		-1.0		-6.2
.125		16.5		16.1		14.1		-2.2		-1.5		-1.4	
	.025		-3.0		-2.6		-2.4		+0.4		-0.7		-0.1
.150		13.5		13.5		11.7		-1.8		-2.2		-1.5	
	.025		-2.3		-2.0		-1.9		0		0		0
.175		11.2		11.5		9.8		-1.8		-2.2		-1.5	
	.025		-1.4		-1.7		-1.5		+0.3		+0.4		0
.200		9.8		9.8		8.3		-1.5		-1.8		-1.5	
	.025		-1.5		-1.5		-1.1		+0.1		0		0
.225		8.3		8.3		7.2		-1.4		-1.8		-1.5	
	.025		-1.4		-1.1		-1.0		+0.3		0		+0.1
.250		6.9		7.2		6.2		-1.1		-1.8		-1.4	

A.5.13

[redacted]

Frame Loads: Blast on panel between head & sill @ each floor level

pressures @ 100% of blast

2nd floor: Load = $\frac{A+B}{2} \times \frac{144}{1000} \times 11 \times 5 = 7.94 \left(\frac{A+B}{2}\right)$ in kips

3rd floor: Load = $7.94 \left(\frac{B+C}{2}\right)$

4th floor: Load = $C \times \frac{144}{1000} \times 11 \times 2 = 3.17\,C$

t	Panel A P_{front}	Panel A P_{rear}	Panel A P_{net} A	Ave $\frac{A+B}{2}$	Panel B P_{front}	Panel B P_{rear}	Panel B P_{net} B	ave $\frac{B+C}{2}$	Panel C P_{front}	Panel C P_{rear}	Panel C P_{net} C	Frame Loads 2nd Fl.	Frame Loads 3rd Fl.	Frame Loads 4th Fl.
0														
.01	42.5	0	42.5	42.0	41.5	0	41.5	40.5	39.5	0	39.5	334	322	125
.02	36.0	0	36.0	34.75	32.5	0	33.5	31.2	28.9	0	28.9	276	248	92
.03	31.1	0	31.1	30.15	29.2	0	29.2	26.8	24.4	0	24.4	239	213	77
.04	24.9	1.7	26.6	26.6	24.9	1.7	26.6	26.5	21.2	5.2	26.4	211	210	84
.05	21.6	6.5	28.1	27.7	21.5	5.8	27.3	26.05	18.8	6.0	24.8	220	207	79
.075	18.0	2.3	20.3	20.15	17.5	2.5	20.0	20.95	15.4	6.5	21.9	160	166	70
.100	13.7	-0.5	13.2	13.45	13.2	0.5	13.7	14.95	11.7	4.5	16.2	107	119	51
.125	10.8	-1.2	9.6	9.75	10.5	-0.6	9.9	10.4	9.2	1.7	10.9	77	83	35
.150	8.8	-1.1	7.7	7.6	8.6	-1.1	7.5	7.1	7.5	-0.8	6.7	60	56	21
.175	7.3	-0.9	6.4	6.05	6.9	-1.2	5.7	5.35	6.1	-1.1	5.0	48	43	16
.200	6.1	-0.9	5.2	5.15	6.2	-1.1	5.1	4.7	5.2	-0.9	4.3	41	37	14
.225	5.2	-0.8	4.4	4.25	5.2	-1.1	4.1	3.85	4.5	-0.9	3.6	34	31	11
.250	4.5	-0.6	3.9	3.65	4.5	-1.1	3.4	3.1	3.7	-0.9	2.8	29	25	9
.275	3.8	-0.6	3.2	2.95	3.8	-1.1	2.7	2.5	3.2	-0.9	2.3	23	20	7
.300	3.2	-0.5	2.7	2.5	3.4	-1.1	2.3	2.15	2.8	-0.8	2.0	20	17	6
.325	2.8	-0.5	2.3	2.1	3.0	-1.1	1.9	1.8	2.4	-0.7	1.7	17	14	5
.350	2.4	-0.5	1.9	1.7	2.6	-1.1	1.5	1.45	2.0	-0.6	1.4	14	11	4
.375	2.0	-0.4	1.6	1.4	2.2	-1.0	1.2	1.2	1.8	-0.6	1.2	11	10	4
.400	1.6	-0.4	1.2	1.0	1.8	-1.0	0.8	0.9	1.6	-0.6	1.0	8	7	3

A5.13

[redacted]

t	F	R	$F-R$	$\sum$	$\sum$	Δx	x	Δx_b	$\Delta x - \Delta x_b$	$\sum$	v_0	$\bar{v}$	v_a	R_{act}
	Avg Force on frame by walls	Avg Net shear for cols above & below		$\sum(F-R) = \sum I \frac{m}{\Delta t} = v_0 \frac{m}{\Delta t}$	$\overline{\sum(F-R)} = \bar{v} \frac{m}{\Delta t}$	$\Delta x = \overline{\sum(F-R)}\Delta t = \bar{v} \frac{m}{\Delta t}$	$x = \sum \Delta x$	Δx_b = chg of defl of col below during same time int.		$\sum(\Delta x - \Delta x_b)$ = total rel displ. of mass with respect to mass below	v_0 = resist. of cols below at end of time int.	$\bar{v}$ = avg resist of cols below during time int.	V_a = avg resist of cols above during time int.	$R_{act} = \bar{v} - v_a$

All signs are automatic with following convention

+F accelerates mass to rt.
+R " " " left (i.e., slows down mass)
+ Δx is chg in deflection to rt. (both abs & rel.)

A.5.14 Concrete Frame with Windows

Frame Analysis

[redacted]

$1/m = .170$

$\frac{\Delta t}{m} = 170 \times 10^{-5}$

$\frac{\Delta t^2}{m} = 1.70 \times 10^{-5}$

t	F	R	F-R	$\mathcal{E}$	$\bar{\mathcal{E}}$	Δx	x	Δx_b	$\Delta x \cdot \Delta x_b$	$\mathcal{E}$	r_o	$\bar{r}_o$	$\bar{r}_n$	R_{act}	F_{act}
0.01	304	-15	319	319	159	.00270	.00270	.00476	-.00206	-.00206	-25	-13	0	-13	+305
0.02	325	-57	382	701	510	.00866	.01136	.01380	-.00514	-.00720	-87	-56	0	-56	+325
0.03	+275	-125	400	1101	901	.01530	.02666	.01975	-.00445	-.01165	-141	-114	0	-114	+270
	+270	-120	390	1091	896	.01522	.02658	.01970	-.00448	-.01168	-141	-114	0	-114	+270
0.04	+250	-130	380	1471	1281	.02180	.04838	.02055	+.00125	-.01043	-126	-133	0	-133	+245
0.05	+242	-80	+322	1793	1632	.02780	.07618	.01890	+.00890	-.00153	-18	-72	0	-72	+240
0.075	+208	+128	+80	718 798	758	08050	.15668	.0433	+.03720	+.03567	+274	+128	0	+128	+187
	+190	+128	+62	780	749	.07960	.15578	.04590	+.03370	+.03217	+274	+128	0	+128	+187
0.100	+109	+266	-157	623	701	.07460	.23038	.05300	+.02460	+.05377	+259	+266	0	+266	+107
	+107	+266	-159	621	701	.07460	.23038	.05320	+.02140	+.05357	+259	+266	0	+266	+107
0.125	+46	+248	-202	419	520	.05530	.28568	.06060	-.00530	+.04827	+215	+237	0	+237	+61
	+60	+241	-181	440	530	.05630	.28668	.05930	-.00300	+.05057	+229	+244	0	+244	+60
0.150	+37	+170	-133	307	373	.03960	.32628	.05310	-.01350	+.03707	+127	+178	0	+178	+60
	+59	+189	-130	310	375	.03990	.32658	.05200	-.01210	+.03847	+136	+183	0	+183	+58

$\frac{\Delta t}{m} = 425 \times 10^{-5}$

$\frac{\Delta t^2}{m} = 10.62 \times 10^{-5}$

A.5.14

[redacted]

t	F	R	$F-R$	ε	$\bar{\varepsilon}$	Δx	x	Δx_b	$\Delta x - \Delta x_b$	ε	r_o	$\bar{F}_o$	$\bar{F}_a$	R_{act}	F_{act}
0.01	755	32	723	723	362	.00476	.00476	.00374	.00102	.00102	38	19.	-13	+32	+752
0.02	821	165	656	1379	1051	.01380	.01856	.00970	.00410	.00512	192	115	-56	171	+820
0.03	+665	+420	+245	1624	1502	.01975	.03831	.01240	.00735	.01247	466	329	-114	+443	+669
	+669	+433	+236	1615	1497	.01970	.03826	.01250	.00720	.01232	460	326	-114	+440	+669
0.04	+581	681	-100	1515	1565	.02055	.05881	.01438	.00617	.01849	658	559	-133	+692	+594
0.05	+588	748	-160	1355	1435	.01890	.07771	.01770	.00120	.01969	670	664	-80	+744	+605
0.075	+492	+522	-30	542 512	527	.04330	.12101	.0459	-.0026	.01709	589	630	+128	+502	+549
	+545	+512	+33	575	558	.04590	.12361	.04990	-.00400	.01569	589	630	+128	+502	+551
0.10	+479	+340	+139	714	644	.05300	.17661	.04400	+.00900	.02469	623	606	+266	+340	+486
	+486	+340	+146	+721	648	.05320	.17681	.04360	+.00960	.02529	623	606	+266	+340	+486
0.125	+388	+355	+33	754	737	.06060	.23741	.02840	+.03220	.05749	582	603	+237	+366	+357
	+361	+361	0	+721	+721	.05930	.23611	.02890	+.03040	.05569	582	603	+244	+359	+359
0.150	+262	+410	-148	+573	+647	.0531	.28921	.0120	+.0411	.09679	579	581	+178	+403	+209
	+212	+392	-180	+541	+631	.05200	.28811	.01332	+.03868	.09437	579	581	+183	+398	+213

$1/m = .1315$

$\frac{\Delta t}{m} = 131.5 \times 10^{-5}$

$\frac{\Delta t^2}{m} = 1.315 \times 10^{-5}$

$\frac{\Delta t}{m} = 329 \times 10^{-5}$

$\frac{\Delta t^2}{m} = 8.22 \times 10^{-5}$

A.5.14

[redacted]

t	F	R	F-R	ε	$\bar{\varepsilon}$	Δx	x	Δx_b	$\Delta x - \Delta x_b$	ε	r_o	$\bar{r}_o$	$\bar{r}_a$	R_{act}	F_{act}
0.01	766	110	656	656	328	.00374	.00374			.00374	264	132	19	113	+768
0.02	861	470	391	1047	852	.00970	.01344			.01344	904	584	115	469	+860
0.03	+706	+626	+80	1127	1087	.01240	.02584			.02584	946	925	329	+596	+709
	+709	+615	+94	1141	1094	.01250	.02594			.02594	946	925	326	+599	+709
0.04	+645	+405	+240	+1380	1260	.01438	.04032			.04032	963	955	559	+396	+637
0.05	+634	+290	+344	+1924	+1552	.01770	.05802			.05802	951	957	664	+293	+603
0.075	+377	+283	+94	690 596x	643	.04590	.10392			.10392	919	935	630	305	+321
	+325	+310	+15	705	698	.04990	.10792			.10792	911	931	630	+301	+319
0.10	+116	+290	-174	531	618	.04400	.15192			.15192	879	895	606	+289	+103
	+103	+289	-186	519	612	.04360	.15152			.15152	879	895	606	+289	+103
0.125	+8	+250	-242	277	398	.02840	.17992			.17992	828	853	603	+250	+25
	+23	+250	-227	292	405	.02890	.18042			.18042	828	853	603	+250	+24
0.150	-6	+241	-247	45	168	.0120	.19242			.19242	817	822	581	+241	+35
	+32	+241	-209	83	187	.01332	.19374			.19374	817	822	581	+241	+31

$1/m = 0.114$

$\frac{\Delta t}{m} = 114 \times 10^{-5}$

$\frac{\Delta t^2}{m} = 1.14 \times 10^{-5}$

$\frac{\Delta t}{m} = 285 \times 10^{-5}$

$\frac{\Delta t^2}{m} = 7.125 \times 10^{-5}$

A.5.14

[redacted]

0.075 to 0.10 (Δt)

3-4 $\Sigma M = 623$ $\Sigma P = 308$

$r_0 = 623 \times 4/9.5 = 262$

65

$-308 \times 2/9.5 \times .05377 = 3$

259

2-3 $E = 10.7 \times 10^3 \times .02469 = 264 > M_{ult}$

$I = 34.7 \times 10^3 \times .02469 = 858 > M_{uH}$

$\Sigma M = 3027$ $\Sigma P = 410$

$3027 \times 2/9.7 = 625$

84.4

$410 \times 2/9.7 \times .02469 = -2$

623

1-2 $\Sigma M = 2125$ $\Sigma P = 522$

$r_0 = 2125 \times .421 = 895$

110

$522 \times 2/9.5 \times .1514 = 16$

879

0.10 to 0.125

3-4 $\Sigma M = 581$ $\Sigma P = 194$

$r_0 = 581 \times 4/9.5 = 249$

40

$194 \times 2/9.5 \times .0483 = -2$

$14{,}320 \times .0053 \times 4/9.5 = -32$

215

Trial #2

249

-2

$14320 \times .00[illegible] \times 4/9.5 = -18$

229

2-3 $\Sigma M = 2834$ $\Sigma P = 296$

$r_0 = 2834 \times 2/9.7 = 585$

61

$296 \times 2/9.7 \times .0575 = 3$

582

1-2 $\Sigma M = 2040$ $\Sigma P = 408$

$r_0 = 2040 \times 4/9.5 = 843$

85

$-408 \times \frac{2}{9.5} \times .1799 = 15$

828

0.125 to 0.150

3-4 $\Sigma M = 541$ $\Sigma P = 122$

$r_0 = 541 \times 4/9.5 = 228$

25.7

$122 \times 2/9.5 \times .03707 = -1$

$14320 \times .0165 \times 4/9.5 = -100$

127

2-3 $\Sigma M = 2828$ $\Sigma P = 214$

$r_0 = 2828 \times 2/9.7 = 583$

44.1

$214 \times 2/9.7 \times .09679 = 4$

579

1-2 $\Sigma M = 1969$ $\Sigma P = 326$

$r_0 = 1969 \times 4/9.5 = 830$

67.3

$-326 \times 2/9.7 \times .1924 = 13$

817

Trial #2

3-4 $r_0 = 228$

$25.7 \times .03777 = -1$

$14320 \times .0151 \times 4/9.5 = -91$

136

A.5.14

[redacted]

0.050 to 0.075

Front

ΔFM_Δ	+74.4	+74.4	-6.0	-6.0	+50.5	+50.5
ΔFM_w	-8.8	+8.8	-9.0	+9.0	-7.4	+7.4
	.5→					←.875
$\Sigma\Delta$ FM	+65.6	+83.2	-15.0	+3.0	+43.1	+57.9
	-15.5		-6.1		-21.7	-6.1
	+5.4	-25.0	+3.3	-15.5	+2.3	+3.3
	+0.9	-2.7	-1.0	+5.4	-1.2	-1.0
	+0.2	-0.5	-0.3	+0.9	+0.4	-0.3
		-0.1	-0.1	+0.2	+0.1	-0.1
		+54.9	-19.2	-6.0	+23.0	+53.7
		-17.8	-17.8	-8.5	-8.5	-40.3
ΔM_1		+37.1	-37.1	-14.5	+14.5	+13.4
						←.5 +1.0
			+0.1		-0.5	+0.1
				+0.2	+0.2	
ΔM_2				+0.2	-0.2	
$\Sigma\Delta M$	0	+37.1	-37.1	-14.3	+14.3	+12.4
M@0.05	+23.9	-8.7	+8.7	-23.2	+23.2	-7.9
M@0.075	+23.9	+28.4	-28.4	-37.5	+37.5	+4.5

Trial #2

Rear

ΔFM_Δ	+74.4	+74.4	-6.0	-6.0	+50.5	+50.5
ΔFM_w	-6.9	+6.9	-4.7	+4.7	+0.3	-0.3
	.5→					←.375
$\Sigma\Delta$	+67.5	+81.3	-10.7	-1.3	+50.8	+50.2
	-15.2		-9.7		-18.8	-9.7
	+5.8	-26.1	+2.9	-15.2	+3.6	+2.9
	+1.0	-2.9	-1.2	+5.8	-1.1	-1.2
	+0.2	-0.5	-0.4	+1.0	+0.5	-0.4
		-0.1	-0.1	+0.2	+0.1	-0.1
		+51.7	-19.2	-9.5	+43.1	+41.7
		-16.2	-16.2	-16.8	-16.8	-31.3
ΔM_1		+35.4	-35.4	-26.3	+26.3	+10.4
		←.5 +4.9	→.5 -4.9			
			-0.6	+2.4		-0.6
			-0.1	+0.3	+0.2	-0.1
				+2.7	+0.2	-0.7
				-1.5	-1.5	+0.5
ΔM_2				+1.2	-1.2	-0.2
$\Sigma\Delta M$	0	+34.5	-30.5	-25.1	+25.1	+10.2
M@0.05	+23.9	+16.5	-16.5	-7.1	+7.1	-2.7
M@0.075	+23.9	+47.0	-47.0	-32.2	+32.2	+7.5
ΣM	+123.2		-145.1		+81.7	
ΣV_M	10.28↑	10.28↓	12.1↓	12.1↑	6.81↑	6.81↓
ΣV_w	13.55↑	27.65↑		31.15↑		17.05↑
F÷8	23.83↑	5.27↑		50.06↑		10.24↑
F	191↑	42↑		401↑		82↑

$\Sigma F = 716$ ✓

$F_4 = (152 + 82) \div 2 + 70 = +187$

$F_3 = (370 + 401) \div 2 + 166 = +551$

$F_2 = (276 + 42) \div 2 + 160 = +319$

A.5.14

Steel Frame with Windows

A.5.15 Columns - P - M & δ_e

To accomplish ~~these~~ resistances: (as found by preliminary analyses) to be required.

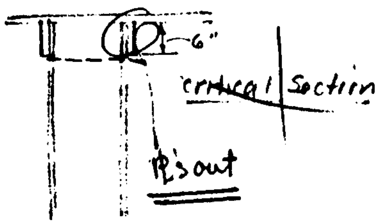
2nd Floor Columns

Ext. 10 WF 112# → avg Myp of ~ 550 KF

~~To avoid yielding at the weld w. this setup, assume a 6" ℞ welded on~~ VOID

length of 1st Floor columns is 9.25

$$\frac{9.25 - 0.50}{8.75}\ \text{FT} \quad \underline{8.83}\ \text{FT}$$



req'd shear in ext. cols = $\frac{550 \times 8}{8.75} = 505^K$

Total req'd = 2200
505
1695 K additional req'd

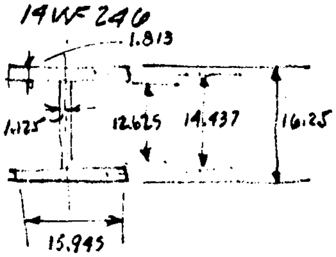
Int. Cols:

req'd Myp = $\frac{1695}{8} \times 8.75 = 1855^{KF}$ avg @ ~ 250 K load

14 WF 237 → 1855 KF
1784 (R-206, ORP 3/10/50)
71 KF additional req'd

Add. approx $\frac{71 \quad 12}{50 \times 14} = 1.22\,\square''$ ∴ 14 WF 246

14 WF 246



$A_{FL} = 29\,\square'' \times 2 = 58\,\square''$
web = 14.2
72.2 □" (72.33 ; ok)

@ P = 710 KF Myp = 1745 KF
@ P = 0 Myp = 1745 + 187 = 1932 KF

1932 11.3 1.39

∴ @P = 250 - $\frac{250}{710} \times 187 = 66^{KF}$
1868 KF O.K. ∴ 14 WF 246 O.K. (probably) (used)

A.5.15

[redacted]

P-M Curves, & Elastic Deflections

(Rev. A - 3/17/50)

[assume joint rotation = .002 radians / floor as previously]

2nd Floor Columns

Interior 14 WF246

I = 3228.9 @ P = 710^k, Myp = 1745^{kF}
Aw = 19.2 @ P = 0, Myp = 1932^{kF}

length of col = 8'-10" = 8.833^{FT} = 106"

Δel @P=0

$$\Delta el_B = \frac{1932 \times 106^2}{6 \times 30{,}000 \times 3228.9} = .0374^{FT}$$

$$\Delta \theta = 8.833 \times .002 = .0176$$

(M=24,864Δ)

$$\Delta v = \frac{1932 \times 2}{\cancel{8.833}} \times \frac{\cancel{8.833 \times 12}}{12{,}000 \times 19.2 \times 12} = .0227$$

$$.0777^{FT}$$

2×.078 = .156
.086
3).242
.081

Δel @ P=710 $= \frac{1745}{1932} \times .0777 = .0702^{FT}$

Exterior 10"WF112

I = 718.7 @P = 0^k M = 610^{kF}
Aw = 6.70 335 548
l = 8'-10" = 8.83^{FT} = 106" 1635 0

Δel @ P=0

$$\Delta el_B = \frac{610 \times 106^2}{6 \times 30{,}000 \times 718.1} = .0530^{FT}$$

$$\Delta \theta = .0176$$

[M=7110Δ]

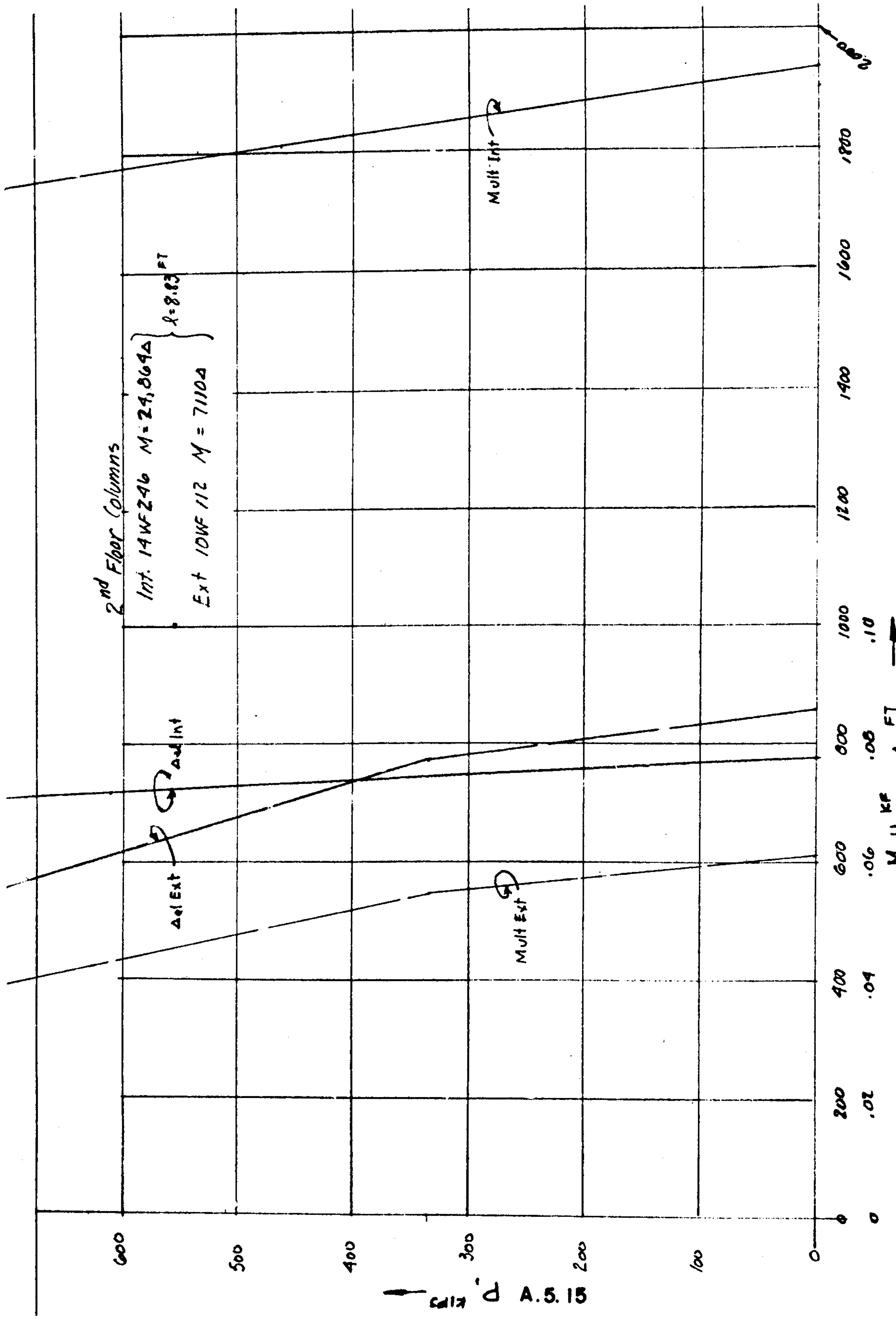
$$\Delta v = \frac{610 \times 2}{6.70 \times 12000} = .0152$$

$$.0858^{FT}$$

Δel@P: 335 $= \frac{548}{610} \times .0858 = .0771$

[Δel@P = 1635 = 0] A.5.15

[redacted]



A.5.15

[redacted]

[Tabulation
This assumes all Roof Slabs are elastic]

See R-101
(DRP 3/2/50)

A.5.16 Loads

t	P_A	B	C	D	Basic Column Loads @ 100% (DL+LL, but no girder shears due to M in Columns, accounted for)												
					Roof				3rd				2nd				
					A^{8}	B^{21}	C^{21}	D^{8}	A^{15}	B^{28}	C^{28}	D^{15}	A^{16}	B^{30}	C^{30}	D^{16}	
0	0	0	0	0	8	21	21	8	23	49	49	23	39	79	79	39	0
.010	162	222	0	0	170	243	21	8	185	271	49	23	201	301	79	39	.010
.020	153	318	110	0	161	339	131	8	176	367	159	23	192	397	189	39	.02
.03	129	358	222	24	132	379	243	32	147	407	271	47	163	437	301	63	.03
.04	90	298	303	95	99	319	324	103	113	347	352	118	129	377	382	134	.04
.05	67	256	325	118	75	277	346	126	90	305	374	141	106	335	404	157	.05
.06	44	222	310	108	52	243	331	116	67	271	359	131	83	301	389	147	.06
.07	24	190	290	97	32	211	311	105	47	239	339	120	63	269	369	136	.07
.08	14	170	282	90	22	191	303	98	37	219	331	113	53	249	361	129	.08
.09	13	162	253	81	21	183	274	89	36	211	302	104	52	241	332	120	.09
.10	12	141	247	73	20	162	268	81	35	190	296	96	51	220	326	112	.10
.125	11	125	190	61	19	146	211	69	34	174	239	84	50	204	269	100	.125
.150	10	81	138	53	18	102	159	61	33	130	187	76	49	160	217	92	.150
.175	9	45	91	44	17	66	112	52	32	94	140	67	48	124	170	83	.75
.200	8	24	65	37	16	45	86	45	31	73	114	60	47	103	144	76	.200
.225	7	21	52	31	15	42	73	39	30	70	101	54	46	100	131	70	.225
.250	6	19	45	26	14	40	66	34	29	68	94	49	45	98	124	65	.250
.275	5	16	37	22	13	37	58	30	28	65	86	45	44	95	116	61	.275
300	4	13	29	16	12	34	50	24	27	62	78	39	43	92	108	55	.300

A.5.16

[redacted]

Loads to Frame @ 100%
See p R-102 (DRP 3/2/50)

t	F_2	F_3	F_{roof}	t
0				0
.01	1500	1444	693	.01
.02	1385	1259	566	.02
.03	1244	1089	492	.03
.04	1056	956	443	.04
.05	911	840	387	.05
.06	796	741	348	.06
.07	696	663	318	.07
.08	600	589	287	.08
.09	519	526	262	.09
.10	444	463	238	.10
.125	361	382	197	.125
.150	250 279	283 296	135 150	.150
.175	222	230	115	.175
.200	168 184	161 181	78 89	.200
.225	151	141	66	.225
.250	110 121	103 115	48 53	.250
.275	99	92	43	.275
.300	79	73	33	.300

FORCES REVISED FOR SMALLER TIME INTERVAL (ST. LINE INTERPOLATION) HJB 3/30/50

t	F_2	F_3	F_{ROOF}	t
.150				
.160	242	253	127	
.170	225	233	116	
.180	209	213	105	
.190	192	193	94	
.20	176	173	83	
.21	162	155	75	
.22	151	144	69	
.23	139	136	63	
.24	128	121	57	
.25	116	109	51	

A.5.16

[redacted]

t	$F+R$	I	Δv	v_0	$\bar{v}$	Δt	t	Δt_b	$\Delta t_b - \Delta t$	Σ	r_0	$\bar{r}$	$\bar{r}_a$	R_c	R_a	F	$F+R_a$	Roof $\frac{1}{m}$ = .270 l = 9.5 Comment $\frac{1}{32}$ = .297'
.010	710	7.10	1.92	1.92	0.96	.0096	.0096	.0128	+.0032	+.0032	+23	+11		+11	+15	695	710	
.020	631	6.31	1.70	85 3.62	2.77	.0277	.0373	.0356	+.0079	.0111	+80	+52		+52	+65	566	631	
.020	621	6.21	1.68	84 3.60	2.76	.0276	.0372	.0359	+.0083	+.0115	+83	+53		+53	+55	566	621	
.030	612	6.12	1.66	83 5.26	4.43	.0443	.0815	.0525	+.0082	+.0197	225 +142	+112		+112	+120	492	612	
.030	606	6.06	1.64	82 5.24	4.42	.0442	.0814	.0528	+.0086	+.0201	228 +145	+114		+114	+114	492	606	
.040	600	6.00	1.62	81 6.86	6.05	.0605	.1419	.0626	+.0021	+.0222	304 +159	+152		+152	+155	445	600	
A.5.17 .040	593	5.93	1.60	6.84	6.04	.0604	.1418	.0612	+.0008	+.0209	+150	+148		+148	+148	445	593	
.050	507	5.07	1.37	68 8.21	7.52	.0752	.2170	.0601	−.0151	+.0058	192 +42	+96		+96	+120	387	507	
.050	487	4.87	1.32	66 8.16	7.50	.0750	.2168	.0602	−.0148	+.0061	194 +44	+97		+97	+100	387	487	
.060	248	2.48	0.64	32 8.80	8.48	.0848	.3016	.0530	.0318	−.0257	141 −185	−70		−70	−100	348	248	
.060	273	2.73	0.74	37 8.90	8.53	.0853	.3021	.0529	−.0324	−.0263	145 −189	−73		−73	−75	348	273	
.070	−27	−0.27	−0.07	8.83	8.87	.0887	.3908	.0459	−.0428	.0691	629 −490	−339		−339	−345	318	−27	Cols at © plastic
.080	−235	−2.35	−0.64	8.19	8.51	.0851	.4759	.0415	−.0436	−.1127	1090 −550	~~−444~~ −520	×	~~−444~~ −520	−522	281	−235	ALL PLASTIC
	−157	−1.57	−0.42	8.41	8.62	.0862	.4770	.0409	.0453	.1144	−550	−444		−444	−444	281	−157	error will cause ~.02" error in δ(ccl) of roof

[redacted]

t	$F+R$	I	Δv	v_0	$\bar{v}$	$\Delta\alpha$	α	$\Delta\alpha_b$	$\Delta\alpha_b - \Delta\alpha$	ε	r_0	$\bar{r}$	$\bar{r}_a$	R_c	R_a	F	$F+R$	3rd Floor $I/m = .181$ $l = 10.ω$ Comment $\frac{l}{32} = .32'$
.010	1414	14.14	2.56	2.56	1.28	.0128	.0128	.0113	.0015	.0015	-20	-10	+11	-21	-30	1444	1414	
.020	1099	10.99	1.99	4.55	3.56	.0356	.0484	.0288	.0068	.0083	-110	-65	+52	-117	-160	1259	1099	
.020	1139	11.39	2.06	62 4.62	3.59	.0359	.0487	.0285	.0074	.0089	138 -118	-69	+53	-122	-120	1259	1139	
.030	689	6.89	1.25	63 5.87	5.25	.0525	.1012	.0334	.0191	.0280	-419 -371	-245	+112	-357	-400	1089	689	
.030	729	7.29	1.32	62 5.94	5.28	.0528	.1015	.0334	.0194	.0283	443 -375	-247	+114	-361	-360	1089	729	
.040	356	3.56	0.64	30 6.58	6.26	.0626	.1641	.0272	.0498	.0781	1411 -1036	-706	+152	-858	-600	956	356	
A5.17 .040	206	2.06	0.37	28 6.31	6.12	.0612	.1627	.0281	.0331	.0614	1187 -812	-593	+148	-751	-750	956	+206	
.050	-330	-3.30	-6.60	5.71	6.01	.0601	.2228	.0223	.0378	.0992	2114 -1302	-1057	+96	-1153	-1170	840	-330	all plastic
.050	-317	-3.17	-0.58	29 5.73	6.02	.0602	.2229	.0225	.0377	.0991	-1302	-1057	+97	-1154	-1157	840	-317	
.060	-459	-4.59	-0.85	43 4.44	5.30	.0530	.2759	.0207	.0323	.1314	-1306	-1304	-70	-1234	-1200	741	-459	
.060	-488	-4.88	-0.88	4.85	5.29	.0529	.2758	.0208	.0321	.1312	-1306	-1309	-73	-1231	-1229	741	-488	
.070	-287	-2.87	-0.52	4.33	4.59	.0459	.3217	.0191	.0268	.1580	-1295	-1300	-339	-961	-950	663	-287	
.080	-199	-1.99	-0.36	3.97	4.15	.0415	.3632	.0159	.0256	.1836	-1295	-1298	-444 -520	-854	-768	589	-199	
	-265	-2.65	-0.48	3.85	4.09	.0409	.3626	.0172	.0237	.1817	-1295	-1298			-754	589	-265	

[redacted]

2nd Floor $\frac{1}{m}$ = .168 l = 8.83

t	F+R	I	Δr	r_0	$\bar{r}$	Δφ	φ	$Δφ_b$	$Δφ_b$ −Δφ	Σ	r_0	$\bar{r}$	$\bar{r}_a$	R_c	R_a	F	F+R	Comment $\frac{l}{32}$ = .278'
.010	1340	13.40	2.26	2.26	1.13	.0113	.0113			.0113	326	163	-10	+153	-160	1500	1340	
.020	735	7.35	1.24	3.50	2.88	.0288	.0401			.0401	1158	-792	-65	-677	-650	1385	735	
.020	710	7.10	1.19	3.45	2.85	.0285	.0398			.0398	1150	-738	-69	-769	-675	1385	710	$\frac{l}{32}$ = .278'
.030	-136	-1.36	-0.23	3.22	3.34	.0334	.0732			.0732	2104	-1627	-245	-1382	-1380	1249	-136	
.030				3.22			.0732			.0732	2104	-1627	-247	-1380	-1380	1249	-136	
.040	-594	-5.94	-1.00	2.22	2.72	.0272	.1004			.1004	2152	-2128	-706	-1422	-1650	1056	-594	all plastic
.040	-494	-4.94	-0.83	41. 2.39	2.81	.0281	.1013			.1013	2152	-2128	-593	-1535	-1550	1056	-494	
.050	-189	-1.89	-0.32	16 2.07	2.23	.0223	.1236			.1236	2132	-2142	+1057	-1085	-1100	911	-189	
.050	-174	-1.74	-0.29	14 2.10	2.25	.0225	.1238			.1238	-2132	2142	-1057	-1085	-1085	911	-174	
.060	-34	-0.34	-0.06	2.04	2.07	.0207	.1445			.1445	-2130	-2131	-1304	-827	-830	796	-34	
.060	-31	-0.31	-0.05	2.05	2.08	.0208	.1446			.1446	-2130	-2131	-1304	-827	-827	796	-31	
.070	-124	-1.24	-0.28	1.77	1.91	.0191	.1637			.1637	-2109	-2120	-1300	-820	-820	696	-124	
.080	-215	-2.15	-0.36	1.41	1.59	.0159	.1796			.1796	-2118	-2119	-1298	-821	-815	600	-215	
	-221	-2.21	-0.37	1.40	1.72	.0172	.1809			.1809	-2118	-2119	-1298	-821	-821	600	-221	

A.5.17

t=.010

Roof $\Delta = .0032$

P=170 all elastic M= 20

243

21

8

442

$20 + 7 = 27 \times \frac{8}{9.5} = 23$

$-442 \times \frac{2 \times .0032}{9.5} = 0$

23

3rd Floor $\Delta = .0015$

P= 185 all elastic M= 18

271

49

23

528

$18 + 8 = 26 \times \frac{8}{10.25} = 20$

$-528 \times \frac{2 \times .0015}{10.25} = 0$

20

2nd Floor $\Delta = .0113$

P= 201 all elastic M= 281

301

79

39

620

$281 + 81 = 362 \times \frac{8}{8.83} = 328$

$-620 \times \frac{2 \times .0113}{8.83} = 2$

326

(Column resistances in elastic range)

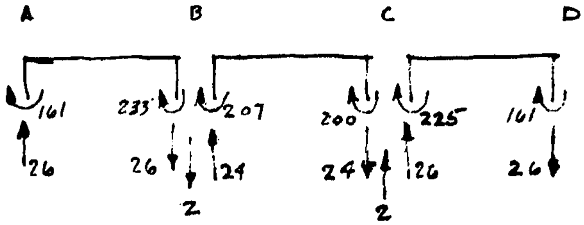
A.5.17

[redacted]

$t = .070$

Roof $\Delta = .0691$

$P = 32$	$M =$ (elastic)	161
211	"	440
311	plastic	425
105	elastic	161
659		1187



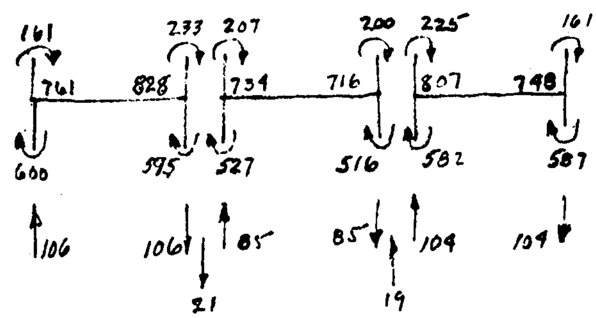
(no appreciable change in axial load or moment @ C)

$\therefore r = 1187 \times \frac{4}{9.5} = 500$

$-659 \times 2 \times \frac{.0691}{9.5} = \frac{-10}{490}$ ←

3^{rd} Floor $\Delta = .1580$

$P = 47$	$M = 600$
239	1122
339	1098
120	587
745	3407



$P = 47 - 132 = 85$ $\quad M = 593$

$239 + 23 = 262$ $\quad 1120$

$339 - 21 = 318$ $\quad 1102$

$120 + 130 = 250$ $\quad 560$

$3375 \times \frac{4}{10.25} = 1318$ ~1% change

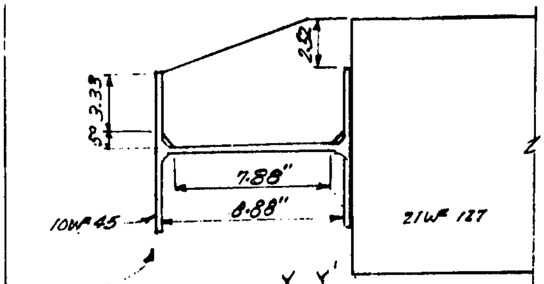
$\left[-795 \times \frac{2 \times .1580}{10.25}\right] = \frac{23}{1295}$ ←

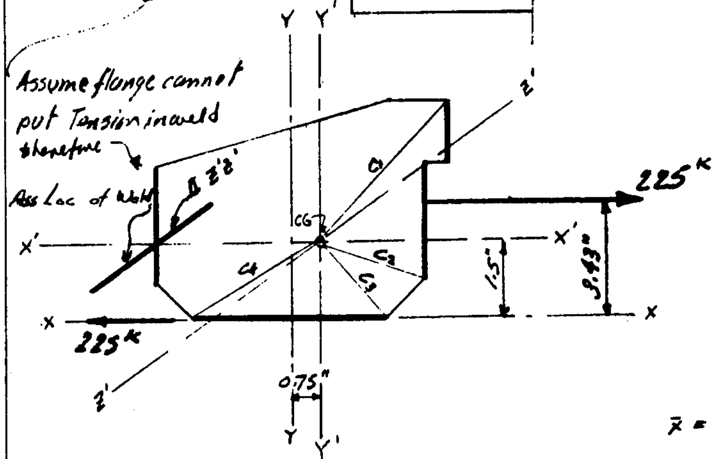
(Column resistances in Plastic Range)

A.5.17

[redacted]

A.5.18 Web Stiffeners





$\bar{y} = 2.52 \times 5.09 = 12.82$

$3.33 \times 2.17 = 7.24$

$3.33 \times \bar{y} = 20.06$

7.88×0

17.06

$\bar{y} = \dfrac{20.06 + 3.33\bar{y}}{17.06}$

$\bar{y} = \dfrac{20.06}{13.73} = 1.50$

$\bar{x} = 2.52 \times 5.06 = 12.73$

3.33

3.33

7.88

17.06

$x' = \dfrac{12.73}{17.06} = 0.75''$

$I_{x'x'}$

$2.52^3/12$	=	1.33
2.52×3.59^2	=	32.3
$3.33^3/12$	=	3.13
$3.33 \times .67^2$	=	1.49
7.88×1.50^2	=	17.7
		55.95

$Ld = 39.1 \times .985 \times 5.85 = 225^K$

$M_{max} = 225 \times 3.43 = 773^{K'}$

$T'' = \dfrac{225}{5.85} = 38.4$

$\sigma'' = \dfrac{225}{7.88} = 28.6$

$I_{y'y'}$

2.52×4.31^2	=	46.8
$+\ 3.33 \times 3.03^2$	=	43.8
3.33×5.19^2	=	89.8
$7.88 \times .75^2$	=	4.4
$7.88^3/12$	=	40.8
		225.6

$I_T = 225.6$
$\underline{56.0}$
281.6

A.5.18

[redacted]

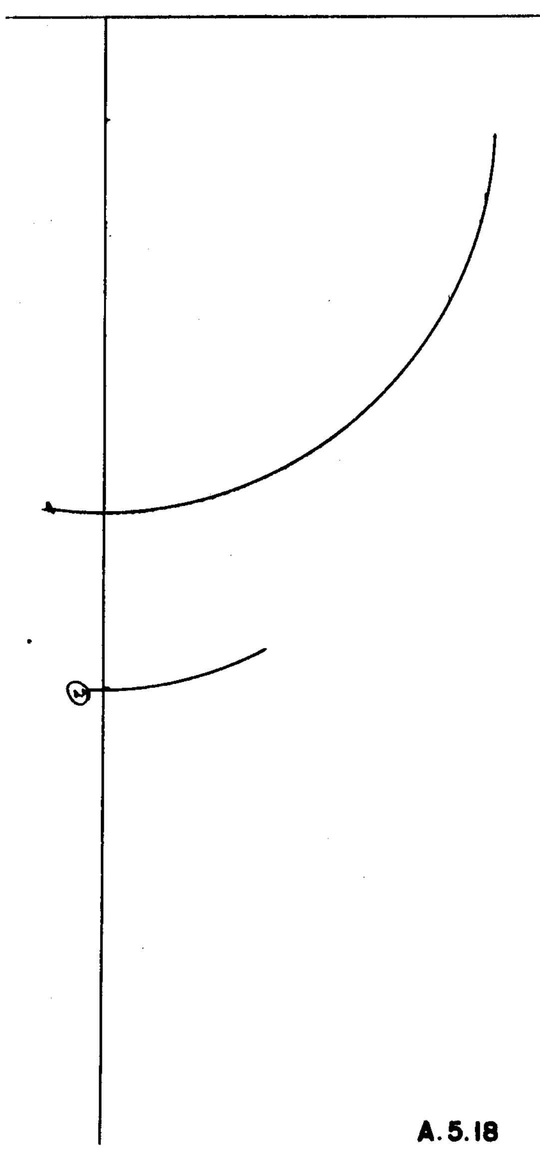
$$C_1 = \sqrt{5.06^2 + 4.03^2} = 6.46$$
$$C_2 = \sqrt{3.69^2 + 1^2} = 3.83$$
$$C_3 = \sqrt{3.19^2 + 1.5^2} = 3.53$$
$$C_4 = \sqrt{4.69^2 + 1.5^2} = 4.93$$

$R_1 = \frac{773}{281.6} \times 6.46 = 17.8$ $\quad t_1 = 38.4 - 11.1 = 27.3$ $\quad v_1 = 14.0$

$R_2 = 10.6$ $\quad t_2 = 38.4 + 2.8 = 40.2$ $\quad v_2 = 10.2$

$R_3 = 9.7$ $\quad t_3 = -8.8$ $\quad v_3 = 28.6 - 4.1 = 24.5$

$R_4 = 13.6$ $\quad t_4 = +12.9$ $\quad v_4 = 28.6 - 4.1 = 24.5$



Max $t = 43^{k/\prime\prime}$

Pl thickness $= \frac{43}{40} = 1.07$

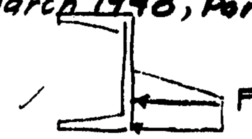
use $1\frac{1}{8}''$ Pl

A.5.18

[redacted]

Refer to: "Composite Construction For I-Beam Bridges"
C.P. SIESS - Proc. A.S.C.E March 1948, Part I

$$F = f_c\left[h_f + \frac{3}{2}t\right]$$



use 6 ⊔ 13.0 $\quad F = 3(.5 + \frac{3}{2}\times .437) = 3.47$ k/"

A.5.19 Shear Connectors

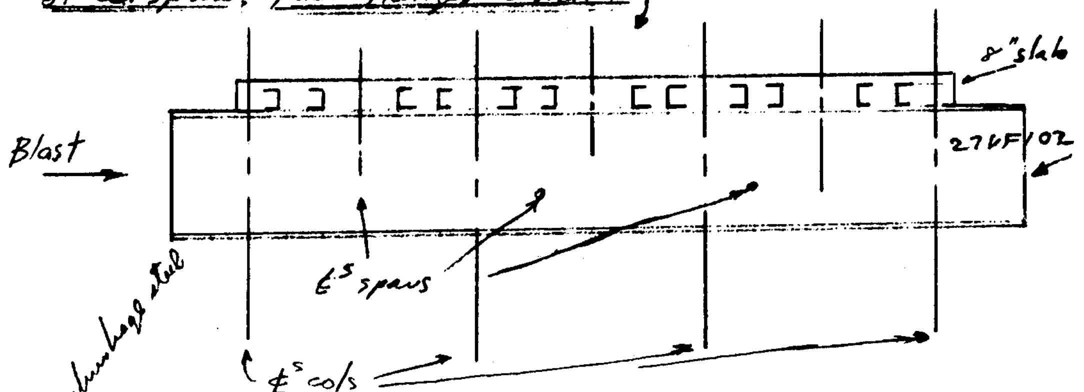
∴ 6⊔13.0 good for $10 \times 3.47 = 34.7^k$

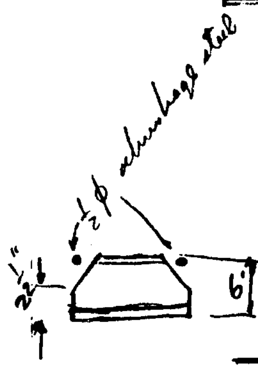
℄ × (req'd space) = $\frac{34.7}{8.53} = 4.07"$ say 4" c.c. @ left

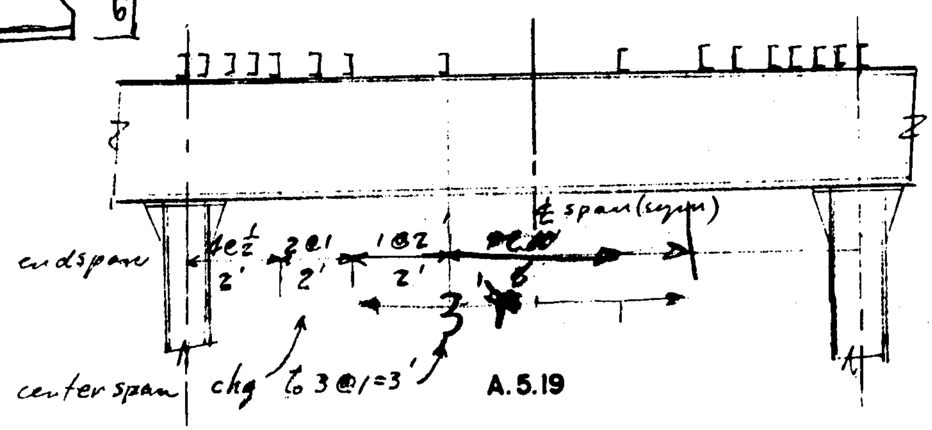
$= \frac{34.7}{6.55} = 4.98"$ (7.27) say 4½ c.c @ rt

$= \frac{34.7}{.994} = 34.9"$ say 3' c.c in center of span

Note: Above spacings involve assumption that all concrete over girder flange contributes to beam action. However, cracks due to extreme plastic strains make part ineffective. Therefore, it seems results obtained on sheets 30-32 are extreme. In this case use min space = 5" c.c at ends & increase spacing to 3'± at center of ea. span. Face flanges as shown.

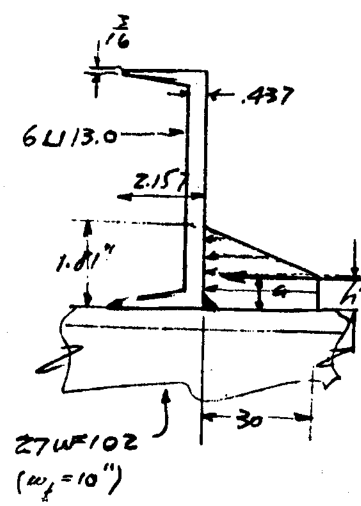






A.5.19

[redacted]



ASSUME FULL LOAD ON ⊔

$h_{ct} = 3t = 1.31'$

$h_f = \frac{.5}{1.81}$

$F_{total} = 34.7^k$

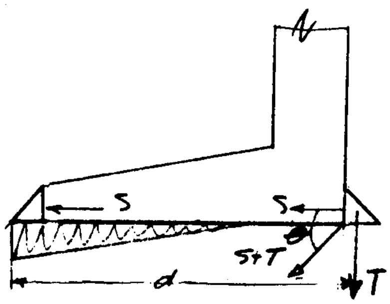
$$30h' + \tfrac{1}{2}\left[(1.81 - h')30\right] = 34.7$$

$$30h' + 27.1 - 15h' = 34.7$$

$$h' = \frac{7.6}{15} = \underline{.506}''$$

$$a = \frac{\overset{19.5}{\tfrac{1}{2}(1.30\times30).939} + \overset{15.2}{(.506\times30).253}}{19.5 + 15.2} = \frac{18.30 + 3.85}{34.7} = \underline{.639}$$

FORCES ON WELD -



use $j = .875$

$d = 2.44''$

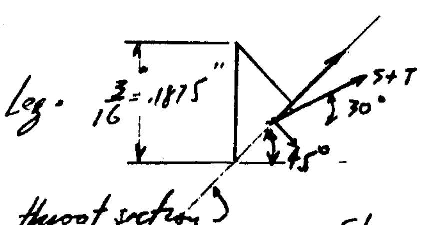
$M = 34.7 \times .634 = 22.0''^k$

$$\therefore T = \frac{M}{jd} = \frac{22.0}{2.14} = 10.3^k$$

$$\text{Let } S = \frac{F_{total}}{2} = 17.4^k$$

$$S+T = \left[17.4^2 + 10.3^2\right]^{\frac{1}{2}} = \underline{20.2}^k$$

$$\tan^{-1}\frac{10.3}{17.4} = 30.6^\circ \ \underline{\text{say } 30^\circ}$$



shear on throat = $20.2 \cos 15^\circ = \underline{19.5}^k$

tension " " = $20.2 \sin 15^\circ = \underline{5}^k$

Shear value, 1" length weld = $30 \times .133 \times 1 = 3.9^k$

tension " " " " = $40 \times$ " " = 5.2^k

Length req'd, shear = $\frac{19.5}{3.9} = \underline{5}$ use $\frac{3}{16}$ ◺ across entire flange

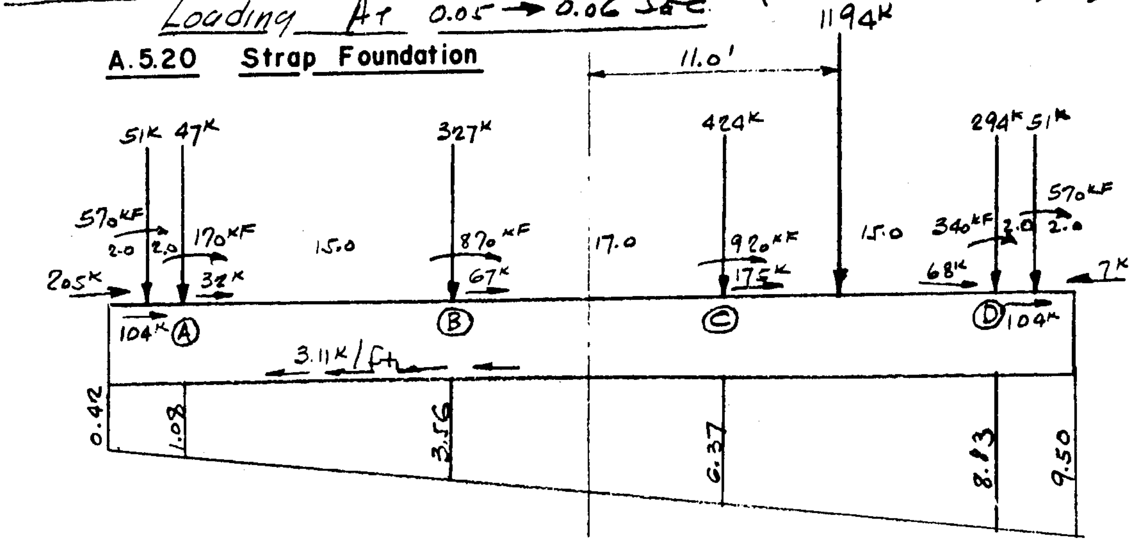
A.5.19

[redacted]

CASE I

Loading At 0.05 → 0.06 Sec (from frame Analysis)

A.5.20 Strap Foundation



51×53 = 2700
47×51 = 2400
327×36 = 11800
424×19 = 8050
294×4 = 1176
51×2 = 102
1194 26228
6432
19796

748×4 = 2992
570
170
870
920
340
570
6432

$\frac{19796}{1194} = 16.5'$

$P = \frac{1194}{240.5} \pm \frac{13150 \times 27.5}{79864} = 4.96 \pm 4.54$

$p = +9.50 \;\&\; +0.42$

MOMENT AT (A)

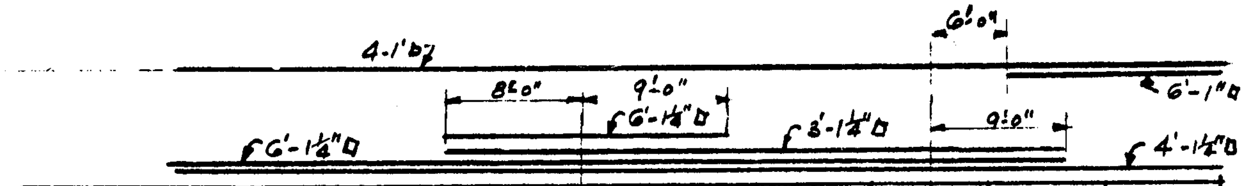
.341×2 = 682
3.11×38×2 = 236
0.42×38×2 = 32
$\frac{0.66}{2}$×38×1.33 = 17
570
1537
51×2 = 102
1435
170
1605

1.08×47×3.5×23.5 = 4175
$\frac{7.25}{2}$×47×3.5×$\frac{2\times47}{3}$ = 19974
883×38×49 = 16441
$\frac{0.67}{2}$×38×49.67 = 632
41222
39564
1658

327×15 = 4905
424×32 = 13568
294×47 = 13818
51×49 = 2499
407×2 = 814
3.11×202.5×2 = 1260
870
920
340
570
39564

A.5.20

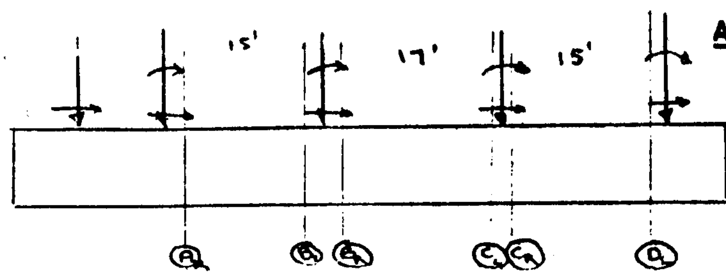
[redacted]



Load Conditions ↓ / M@→	End	A	Center	B	Center	C	Center	D	End
Case I		-1658	-1395	-2940	+681	-2003	-1065	+550	
Case II		-2389	-2221	-4234	-1711	-1677	+304	+788	
Case III		-1183	-907	-2182	-866	-965	+608	+823	
+Resist M		743	743	743	743	743	743	1800	
-Resist M (of Steel shown above)		2720	2720	4280	3120	3120	1195	1195	

A.5.20

[redacted]



A.5.21 Mat Foundation

$P_c = .85 \times 114" \times 3^{1/2}" = 290^k$ $\quad d = 57.5$ $\quad A_s = 40$ ksi

$Z = 27.5$ $\quad A_s = 50$ ksi

PT	$M^{'k}$	N^k	e_c''	Z''	e_s''	$M_s^{'k}$	ac''	c''	a''	P_c^k	P_s^k	$A_s^{\square''}$
A_R	-103 -776	228 368.4	5.4" 25.2	27.5	32.9 52.7	7500 19,400	26 67	56.91 55.5	1.18 ~~1.24~~	342 ~~351~~	— —	— 1.54 1.90" ~~1.17~~
B_L	-925 +691	228 84.8	48.7" 98.3"		76.2 125.8	17,400 10,700	60 37	56.98 57.18	1.05 ~~1.08~~ .64 ~~.67~~	305 ~~314~~ 186 ~~194~~	77 ~~86~~ 101 ~~109~~	-1.93 ~~2.15~~ +2.54 ~~2.73~~ [illegible]
B_R	-3861 -2053	648 476.8	71.7 51.7		99.2 79.2	64,300 37,800	222 130	55.5 56.34	4.00 2.32 ~~2.34~~	1160 673 ~~[illegible]~~	512 196 ~~205~~	-12.8 -10.2
C_L	-43 +2269	648 59.8	— 455		— 482.5	28,900	100	56.62	1.76 ~~1.80~~	510 ~~522~~	648 114x60 450 ~~460~~	9.0 ~~9.7~~ +~~10.5~~" 11.3
C_R	-3005 -585	1072 465.8	34 15"		61.5 42.5	66,000 19,800	228 68	55.43 56.90	4.14 ~~4.11~~ 1.20 ~~1.23~~	1200 ~~119~~ 349 ~~[illegible]~~	128 178 —	-3.20 ~~2.45~~ -~~2.36~~ 2.56
D_L	+1248 +90	527 13.7	28.4 790		55.9 817.5	29,500 11,200	102 3856	56.6 57.16	1.80 ~~1.84~~ .68 ~~.70~~	520 ~~534~~ 197 ~~[illegible]~~	183 ~~188~~	4.6" ~~4.7~~ ~~13.76~~ 3.66
D_R	+453 +47	641 136	8.5" 4.15"		— 31.65	— 4300	—		—	—	—	

$l = 9.5'$

$$\frac{2.73}{9.5} = .28 \, \square''/'$$

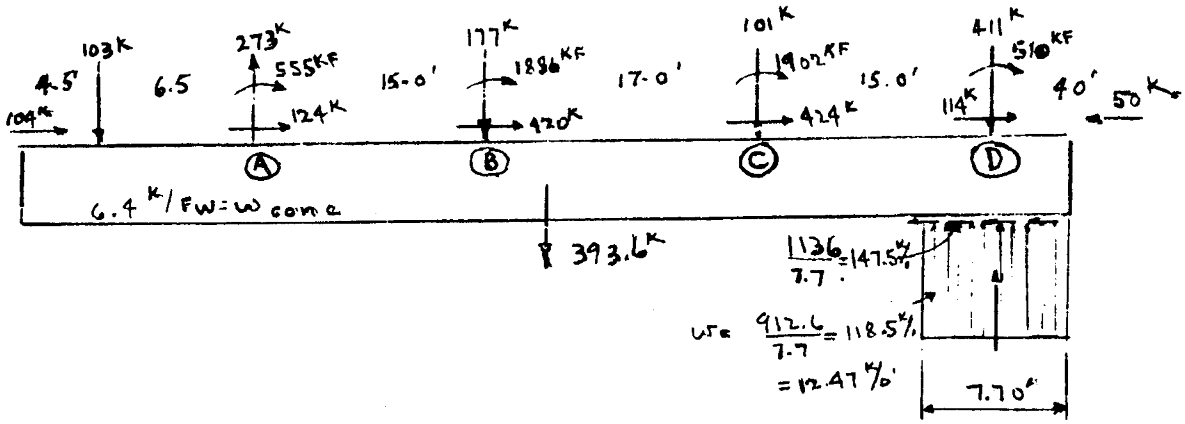
$$\frac{12.8}{9.5} = 1.35 \, \square''/'$$

$$\frac{11.5}{9.5} = 1.21 \, \square''/'$$

$$\frac{4.7}{9.5} = .5 \, \square''/'$$

A5.21

CASE I — AT 0.2 Sec (Loading from Frame Analysis)



1136 × 5 = 5680
555
1886
1902
510
273 × 51 = 13923
24456

177 × 36 = 6372
101 × 19 = 1919
411 × 4 = 1644
103 × 57.5 = 5923
393.6 × 30.75 = 12103
27961
− 24456
3505

$\frac{3505}{912.6} = 3.85'$
× 2
7.70'

CK $\frac{52621}{912.6} = 57.65$ — 61.50, 57.65, 3.85 ck ✓

MOMENT @ A

103 × 6.5 = 669.5
6.4 × $\frac{10.5^2}{2}$ = 352.8
1022.3

228 × 2.5 = 570
555
1125
1022.3
102.7

M = 102.7 KF

6.4 × $\frac{54^2}{2}$ = 8323
177 × 15 = 2655
101 × 32 = 3232
411 × 47 = 19317
= 1886
= 1902
= 510
2044 × 2.5 = 5110
42935

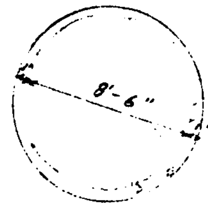
912.6 × 47.15 = 43029
42935
94 KF

M = 103 KF

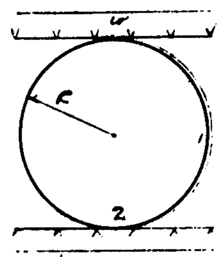
A.5.21

[redacted]

A.5.22 Circular Section



$$M_1 = -M_2 = \frac{wR^2}{4}$$



$\Delta_1 = -\Delta_2$

$R = 4'-9"$

$w = 60\ lb/in/in.$

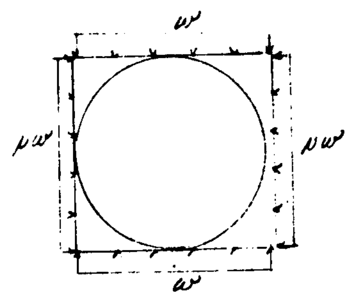
effective p : $\frac{1}{3}$, $\frac{1}{2}$, $\frac{3}{4}$

$$M = \frac{60 \times 12 \times (4.75)^2 \times 12}{4}$$

$f_s' = 50,000\ psi$ $f_c' = 3,000\ psi$

$M = 48,735$ ft-lb per ft.

impact $f_s' = 65,000\ psi$ $f_c' = 3,300\ psi$



For $p = \frac{1}{3}$ $f_s' = 50,000\ psi$
$f_c' = 3,000\ psi$

$$M = 48,735\left(1 - \frac{1}{3}\right) = 33,490 \text{ ft-lb per ft} = 390,000 \text{ in-lb/ft}$$

$M = Rbd^2$ $32,490 \times 2 = 333 \times 12 \times d^2$ $d^2 = 925"$ $d \doteq 10$

For $d = 10$ $R = \frac{32,490 \times 12}{12 \times 100} = 325$ $p = 0.007$

$$A_s = pbd = 0.007 \times 12 \times 10 = 0.84\ in^2$$

$\frac{3}{4}"\phi$ @ $4" = 1.35\ in^2$

A.5.22

[redacted]

8" thick wall

$$\Delta = \frac{1}{12}\frac{wR^4}{EI} = \frac{60\times(55)^4}{12\cdot 3\cdot 10^6\cdot \frac{1}{12}(1)(8)^3}$$

$$= \frac{\cancel{60}^{\cancel{20}\,10} \times 3025 \times 5025}{\cancel{12}\cdot 3\, 10^6\cdot \frac{1}{12}\cdot \cancel{512}_{256}} = \frac{91.51}{256} = 0.357"$$

$k = 500, 200, 100, 50, 25$

for $k = 500$

try 30 psi	$\Delta = 0.179"$	→ 89.5 psi
40	$\Delta = 0.119$	→ 59.5
45	$\Delta = 0.0894$	→ 44.7

$60 - 45 = 15$

$\mu = 1 - \frac{15}{60} = 1 - \frac{1}{4} = 0.75$

$k = 200$

try 30 psi	$\Delta = 0.179"$	→ 35.8 psi
40	$\Delta = 0.119$	→ 23.8
33	$\Delta = 0.161$	→ 32.2
32.5	$\Delta = 0.164$	→ 32.8
32.7	$\Delta = 0.162$	→ 32.4

$60 - 32.6 = 27.4$

$\mu = 1 - \frac{27.4}{60} = 1 - 0.46 = 0.54$

$k = 100$

try 30 psi	$\Delta = 0.179"$	→ 17.9 psi
20	$\Delta = 0.338$	→ 23.8
35	$\Delta = 0.207$	→ 20.7
22.5	$\Delta = 0.223$	→ 22.3
22.4	$\Delta = 0.225$	→ 22.5

$60 - 22.5 = 37.5$

$\mu = 1 - \frac{37.5}{60} = 1 - 0.63 = 0.36$

A.5.22

→ $3/4"\phi$ @ 4" $t = 7 + 3 = 10"$

$f_s' = 65,000$ psi
$f_c' = 3300$ psi

$$d' = \frac{438,610}{1.32 \times 65,000} = 5.11"$$ use $5\frac{1}{2}"$

→ $3/4"\phi$ @ 4" $t = 5\frac{1}{2} + 3 = 8\frac{1}{2}"$

For $\mu = 0.15$

$f_s' = 50,000$ psi
$f_c' = 3000$ psi

$M = 48,735(1 - 0.15) = 41,425$ ft-lb per ft
$= 497,100$ in-lb per ft

$$d' = \frac{497,100}{1.32 \times 50,000} = 7.53"$$ use 8"

→ $3/4"\phi$ @ 4" $t = 8 + 3 = 11"$

$f_s' = 65,000$ psi
$f_c' = 3300$ psi

$$d' = \frac{497,100}{1.32 \times 65,000} = 5.8"$$ use 6"

→ $3/4"\phi$ @ 4" $t = 6 + 3 = 9"$

Spacing 4" o.c.

μ	$f_s' = 50,000$ psi	$f_s' = 65,000$ psi
0	$3/4"\phi$ $t = 12"$	$3/4"\phi$ $t = 10"$
0.15	$3/4"\phi$ $t = 11"$	$3/4"\phi$ $t = 9"$
0.25	$3/4"\phi$ $t = 10"$	$3/4"\phi$ $t = 8\frac{1}{2}"$
0.33	$3/4"\phi$ $t = 9"$	$3/4"\phi$ $t = 8"$
0.50	$3/4"\phi$ $t = 7\frac{1}{2}"$ $5/8"\phi$ $t = 9\frac{1}{2}"$	$5/8"\phi$ $t = 8"$
0.75	$5/8"\phi$ $t = 6\frac{1}{2}"$	$5/8"\phi$ $t = 5\frac{1}{2}"$

$$\Delta = \frac{w\ell^4}{12EI} \quad t = 8"$$

k	μ
500	0.75
200	0.54
100	0.36
50	0.23
25	0.13

A.5.22

[redacted]

APPENDIX 6

TECHNICAL SPECIFICATIONS

CONTENTS

[redacted]

SECTION 1

EXCAVATION, FILLING, AND BACKFILLING

1-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances and materials, and in performing all operations in connection with the excavation, filling, and backfilling, complete, in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

1-02.(*) CLASSIFICATION OF EXCAVATION: Excavation shall be classified as follows:

a. Earth Excavation: Earth excavation shall consist of material not having the properties to be classified as rock excavation.

b. Rock Excavation: Rock excavation shall consist of material which makes its removal and handling by power shovel impractical, except after preliminary breaking by the use of explosives. The Contracting Officer shall be the judge as to the necessity for the use of explosives.

1-03. EXCAVATION:

a. General: The area indicated on the drawings shall be cleared of all natural obstructions and other items which will interfere with the construction operations. The excavation shall conform to the dimensions and elevations indicated on the drawings for the buildings, except as specified below, and all work incidental thereto.

As soon as the site has been established borings and probings shall be made so as to determine the characteristics of the soil, to locate any large voids or water pockets, and to establish the depth of the bed rock. The foundation bearing shall be as indicated on drawing No. 1, i.e., 10 tons p.s.f. under the rear portion of the building and 3 tons p.s.f. under the front portion of the building. Four load tests shall be made along the line of the rear footings, one at each of buildings 1 and 7 plus two intermediate load tests between buildings 1 and 7. A stress strain load curve shall be drawn for each load test point increasing the load in two ton increments from zero to either 12 tons p.s.f. or the yield point of the soil, whichever is less. For each increment the load shall remain in place until there is no settlement in a two-hour period. The yield point is defined as the load under which the strain continues to increase under the applied load. If the load tests indicate that the yield point of the soil under the rear wall footings is less than 10 tons per square foot the foundations materials shall either be removed and replaced by a lean (1:8) mix concrete, or consolidated by injection of a neat cement grout. If suitable bearings are encountered at different elevations from those indicated on the drawings the footing depth may be increased up to 12 inches. In no case, however,

(*) Inapplicable if construction is carried out under a cost-plus-fixed-fee (CPFF) Contract.

[redacted]

shall the top of the footing be changed or the depth of the footing be decreased from that shown on the drawings.

All pockets beneath the footing areas shall be filled with grou~~p~~t of the above mix.

Where the excavation under the first floor slabs of Buildings No. 2 to No. 6 inclusive is made below the elevations shown on the drawings, the subgrade shall be restored to the proper elevation in accordance with the procedure hereinafter specified under 1-05. Backfilling.

Excavation shall extend a sufficient distance from walls and footings to allow for placing and removal of forms, and for inspection, except where the concrete for walls and footings is authorized to be deposited directly against excavated surfaces. Undercutting will not be permitted. Suitable excavated material which is required for fill under slabs, shall be separately stockpiled as directed by the Contracting Officer.

b. Drainage in Vicinity of Buildings and Other Structures: The contractor shall control the grading in the vicinity of buildings and other structures so that the surface of the ground will be properly sloped to prevent water from running into the excavated areas. Any water which accumulates in the excavation shall be removed promptly.

c. Shoring: Such shoring as may be required during excavation shall be installed to protect the banks, adjacent paving, structures and test utilities.

d. Excess Material: Excess material from excavation, not required for fill or backfill, shall be wasted. Wasted material shall be spread and leveled or graded, as directed by the Contracting Officer.

1-04. FILLING: Where concrete slabs are placed on earth, undesirable material as determined by the Contracting Officer, shall be removed. Where fill is required to raise the subgrade for concrete slabs to the elevations as indicated on the drawings or as required by the Contracting Officer, such fill shall consist of broken stone, sand, gravel, or other material approved by the Contracting Officer.

1-05. BACKFILLING: After completion of foundation footings and walls below the elevation of the final grades, and prior to backfilling, all forms shall be removed and the excavation shall be cleaned of all trash and debris. Backfill shall be placed in horizontal layers not in excess of 9 inches in thickness, and shall have a moisture content such that the required degree of compaction may be obtained. Compaction shall be equal to that secured by 5 passes of a loaded 3 cubic yard dump truck with dual tires or as approved by the Contracting Officer.

[redacted]

1-06.(*) MEASUREMENT: The unit of measurement for excavation shall be a cubic yard. The yardage used for calculating the amounts of excavation, fill, and backfill shall be based on the volume excavated within the neat lines of foundation walls, piers,or other subgrade construction, from the elevation of the existing surface, as determined by the Contracting Officer, and the bottom of excavation.

(*) Inapplicable if construction is carried out under a cost-plus-fixed-fee (CPFF) contract.

SECTION 2

CONCRETE

2-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances and materials, and in performing all operations in connection with the installation of concrete work, complete, in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

2-02. APPLICABLE SPECIFICATIONS: The following specifications form a part of this specification:

<u>a</u>. <u>Federal Specifications</u>:

QQ-B-71a.	Bars: Reinforcement (for Concrete)
SS-C-192.	Cements; Portland
UU-P-264.	Paper, Kraft, Concrete-Curing, Waterproofed.
DDD-M-148.	Mat; Cotton, for Concrete-Curing

<u>b</u>. <u>Joint Army-Navy Specification</u>:

Jan-P-66.	Plywood, Flat Panel

<u>c</u>. <u>American Society for Testing Materials: Standard Designations</u>:

A 82-34	Specification for Cold Drawn Steel Wire for Concrete Reinforcement
A 185-37	Specification for Welded Steel Wire Fabric for Concrete Reinforcement.
C 31-44	Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field
C 39-44	Method of Test for Compressive Strength of Molded Concrete Cylinders.
C 40-33	Method of Test for Organic Impurities in Sands for Concrete
C 114-44	Standard Methods of Chemical Analysis of Portland Cement
C 143-39	Method of Slump Test for Consistency of Portland Cement Concrete

[redacted]

2-03. MATERIALS:

a. Acclerating Agent: No accelerating agent shall be used except by the written consent of the Contracting Officer.

b. Aggregate: Aggregate shall conform to the requirements of the Contracting Officer.

c. Cement: Cement shall be used in the sequence of shipments received, unless otherwise directed by the Contracting Officer.

(1) Portland Cement: Portland Cement, except high-early-strength portland cement, shall conform to the requirements of Federal Specifications type I, Ia, II, or IIa as directed by the Contracting Officer.

(2) High-Early-Strength Portland Cement: High-early-strength portland cement, where use is approved by the Contracting Officer, shall conform to the requirements of Federal Specification SS-C-192 type III.

d. Curing Treatments:

(1) Kraft Paper: Kraft paper shall conform to the requirements of Federal Specification UU-P-264, and shall be treatod so as to possess a strength when wet that will afford good resistance to scuffing and shrinkage.

(2) Quilts: Quilts shall conform to the requirements of Federal Specification DDD-M-148.

(3) Burlap: Burlap shall be of commercial quality, and when used shall be in not less than 2 layers.

(4) Water or Membrane-Compound Curing: Curing may be accomplished by effective protection of all exposed surfaces against moisture loss as specified above or by either of the methods under this heading, as ordered by the Contracting Officer.

e. Drainage Fill: Fill under the concrete floor slab shall be broken stone, gravel, or other coarse material satisfactory to the Contracting Officer. Kraft paper over drainage fill shall conform to the requirements of federal Specifications UU-P-264.

f. Forms: Forms shall be of wood, metal, or other material as approved by the Contracting Officer

(1) Wood Forms:

(a) Concrete Surfaces: Wood forms for concrete surfaces shall be No. 2. common or better lumber.

[redacted]

(2) Plywood: Plywood for forms shall conform to the requirements of Joint Army-Navy Specifications Jan-P-66 Commercial Douglas fir moisture resistant, concrete form plywood, not less than 5 ply and at least 9/16 inch thick.

(3) Metal Forms: Metal forms shall be of a type approved by the Contracting Officer, that will produce surfaces equal to those specified for wood forms.

(4) Form Oil: Form oil shall be a non-staining mineral oil.

(5) Form Ties: Form ties shall be of design approved by the Contracting Officer. Ties shall be adjustable in length and free of devices which will leave a hole of depression back of the exposed surface of the concrete larger than 7/8 inch in diameter.

g. Reinforcement Bars: Reinforcement bars for concrete shall conform to the requirements of Federal Specifications QQ-B-71a, type B, deformed, grade 2, intermediate billet. The deformations shall be a high-bond type, conforming to the requirements of A.S.T.M. Specifications A305-49 Types and grades of steel for specific purposes shall be as specified below.

(1) Stirrups: Stirrups shall be of type B, deformed, grade 2 intermediate billet.

(2) Column Spirals: Column spirals shall be plain cold-drawn wire conforming to the requirements of American Society for Testing Materials Standard Specification A82-34.

(3) Mill Reports: Certified copies of mill reports shall accompany all deliveries of reinforcing steel.

h. Mesh Reinforcement: Mesh reinforcement for slabs shall conform to the requirements of A.S.T.M. Specification A185-37.

i. Water: Water shall be clean, fresh, free from oil, acids, alkali, vegetable, sewage, organic or other deleterious matter.

2-04. ADMIXTURES:

a. Admixtures: Admixtures other than air entraining admixtures shall be used only with the written consent of the Contracting Officer.

b. Tests: Tests of admixtures will be made by the government in accordance with applicable Federal or A.S.T.M. Specifications or as otherwise prescribed by the Contracting Officer.

[redacted]

2-05. SAMPLES AND TESTING:

a. General: Testing of the aggregate and reinforcement shall be the responsibility of the contractor. The testing equipment and methods shall be approved by the Contracting Officer. Cement will be tested at the mill by the Bureau of Standards at the Contractor's expense. No material shall be used in the concrete until certified copies of the results of tests are approved in writing by the Contracting Officer. Samples of concrete for strength tests of end items shall be provided and stored by the contractor when and as directed by the Contracting Officer.

b. Aggregate: Aggregate shall be tested as prescribed in Federal Specification SS-A-281 or as directed by the Contracting Officer. In addition fine aggregate shall be tested for organic impurities in accordance with American Society for Testing Materials Standard Method of Test C 40-33.

c. Reinforcement

Ten sample reinforcing bars of 12 inch length shall be taken from each building for each of the following size groups: 5/8 inch or less, 5/8 inch to 1 inch, and over 1 inch size. The ten samples shall be selected so as to represent a specimen from the wall steel, the floor slab steel, the roof slab steel, the column steel, and foundation steel. Each specimen shall be securely tagged so as to identify the source of the sample with respect to the building and shall be forwarded by the contractor to the testing laboratory as directed by the Contracting Officer.

d. Concrete:

(1) Cylinders (see table below): Three test cylinders shall be made for each floor and roof slab of each building (three cylinders per floor) and each wall and column system (three cylinders between each floor) for each building. One cylinder of each set of three shall be tested for strength by the contractor at 7 days (if regular portland cement is used) or at 3 days (if high-early cement is used). The second cylinder of each set shall be tested for strength by the contractor at 28 days or 7 days for regular or high-early portland cement respectively. The third cylinder of each set shall be forwarded by the contractor to a testing laboratory as directed by the Contracting Officer.

(2) Test Beams: Four test beams for each building shall be fabricated by the contractor in accordance with the details of the test beam as shown on the drawings. One test beam shall be made from the foundation, the second floor, the third floor, and the roof concrete. The contractor shall take pains to maintain the dimensions shown on the drawings. The test specimens shall be cured to the same extent as the

[redacted]

prototype structure. Each test beam shall be tagged, identified and forwarded as provided hereinbefore under Section 2-05, c. Reinforcement.

TABLE SHOWING MINIMUM COMPRESSIVE STRENGTH AT 7 DAYS AND 28 DAYS:

Type of Concrete	Compressive strength for design purposes	Average for any 5 consecutive cylinders		Any one cylinder	
		Pounds per sq. in.		Pounds per sq. in.	
		at 7 days	at 28 days	at 7 days	at 28 days
A	3000	2400	3600	2000	3000

Concrete made with high-early strength cement shall have a 7 day compressive strength equal to the specified minimum 28 day compressive strength for concrete of the type specified made with ordinary portland cement.

2-06. STORAGE: Storage accomodations shall be subject to the approval of the Contracting Officer, and shall be such as to permit easy access for inspection and definite identification of each shipment in accordance with the report of tests.

a. Cement: Immediately upon receipt at the site of the work, cement shall be stored in a dry weathertight, properly ventilated structure with adequate provision for the prevention of absorption of moisture.

b. Aggregate: Aggregate shall be stored to avoid the inclusion of any foreign matter in the aggregate and resulting concrete. Storage piles shall be maintained in a manner that will afford good drainage, prevent segregation of particle size and preserve the aggregate gradation. Sufficient live storage shall be maintained at all times to allow placement of concrete at the required rate and to permit time for and inspection.

2-07. TYPES OF CONCRETE AND USAGE:

a. Type A Concrete: Type A concrete shall be used for all concrete shown on the drawings or specified, unless otherwise prescribed by the Contracting Officer.

2-08. PROPORTIONING OF CONCRETE: All materials used in the concrete shall be proportioned by weight and in a manner directed or approved by the Contracting Officer, Prior to placing the concrete in the structure, a sufficient number of trial design batches shall be made and cylinders thereof tested to determine the mix to be used.

a. Measurements:

(1) Cement: A one cubic foot bag of portland cement will be considered as 94 pounds in weight.

(2) Water: One gallon of water will be considered as 8.33 pounds. No increase in the maximum water content as determined in the approval trial batches will be permitted.

(3) Aggregate: The maximum amount of the maximum size of coarse aggregate economically available and placeable as compatible with the type and character of the structure, shall be used.

b. Corrective Additions: Corrective additions to remedy the deficiencies in the aggregate gradations may be used, subject to approval by the Contracting Officer

c. Control: The proportions and exact amounts of all material entering into the concrete shall be as determined in the approved trial mixes. All necessary equipment shall be provided to determine and control the actual amounts of materials entering into each concrete mix. The concrete slump at all times shall be kept between $1\frac{1}{2}$ and 4 inches.

2-09. BATCHING AND MIXING CONCRETE: Concrete shall be mixed by a mechanical batch type mixing plant. The rated capacity of any individual mixer shall be 1/2 cubic yard or more. The mixing plant shall be provided with adequate facilities for accurate measurement and control of each of the materials entering the mixer, and for changing the proportions to conform to varying conditions of the work. The mixing plant assembly shall include adequate provisions for the inspection of operations at all times. The plant and its location shall be subject to approval by the Contracting Officer.

a. Batching Unit: Each batching unit shall be supplied with the following items:

(1) Weighing Unit: A weighing unit shall be provided for each type of material to indicate the scale load at convenient stages of the weighing operation. The weighing units shall be checked at such times as directed by and in the presence of the Contracting Officer and required adjustments shall be made before further use of the device.

(2) Water Mechanism: The water mechanism shall be tight with interlocked valves such that the discharge valve cannot be opened before the filling valve is fully closed. This mechanism shall be fitted with a graduated gauge.

[redacted]

(3) Discharge Gate: The batcher discharge gate shall be so controlled as to permit a ribboning and mixing of the cement with the aggregate. Delivery of materials from the batching equipment to the mixer shall be accurate within the following limits:

Material	Percent by Weight
Cement	1
Water	1
Fine aggregate	2
Coarse aggregate	2 or 3

b. Mixing Unit:

(1) Operation: Mixers shall not be charged in excess of rated capacity, nor be operated in excess of the rated speed. Excessive mixing, requiring the addition of water to preserve the required consistency will not be permitted. The entire batch shall be discharged before recharging.

(2) Mixing Time: Mixing time shall be measured from the instant when the water is introduced into the drum containing all solids. All mixing water shall be introduced before 1/4 of the mixing time has elapsed. Mixing time for mixers of 1 cubic yard or less shall be 1-1/4 minutes: for mixers larger than 1 cubic yard capacity, the mixing time shall be increased 15 seconds for each additional half-cubic yard capacity or fraction thereof.

(3) Discharge Lock: Unless waived by the Contracting Officer, a lock to lock the discharge mechanism until the required mixing time has elapsed, shall be provided on each mixer.

2-10. PREPARATION FOR PLACING: Water shall be removed from excavations before concrete is deposited. Any flow of water shall be diverted through proper side drains and shall be removed by methods which will avoid washing over the freshly deposited concrete. Hardened concrete, wood chips, and shavings and other debris, shall be removed from the interior of the forms, and all hardened concrete and foreign materials shall be removed from the inner surfaces of the mixing and conveying equipment. Wood forms shall be oiled or wetted with water in advance of pouring so that joints will tighten and prevent seepage of cement grout from the mix. Reinforcement shall be secured in position, inspected and approved by the Contracting Officer before starting the pouring of concrete. Runways, or other means approved by the Contracting Officer, shall be provided for wheeled equipment to convey the concrete to the points of deposit. The equipment used to deposit concrete shall not be wheeled over the reinforcement, nor the runways be supported on reinforcement.

2-11. CONSTRUCTION JOINTS: Concrete shall be placed continuously so that the unit of operation will be monolithic in construction. At

least 48 hours shall elapse between the casting of adjoining units unless this requirement is waived by the Contracting Officer. Lifts shall terminate at such levels as are indicated on the drawings, or will conform to structural requirements or as directed by the Contracting Officer. Special provision shall be made for jointing successive pours as detailed on drawings or required by the Contracting Officer. Where the drawings show the floor slab and columns and walls below the slab to be poured integrally, the slab concrete shall not be placed until at least two hours after placing the columns and walls below the floor slab. Any additional construction joints desired by the contractor shall be submitted for the approval of the Contracting Officer.

2-12. CONVEYING: Concrete shall be conveyed from mixer to forms as rapidly as practicable by a method which will prevent segregation or loss of ingredients.

2-13. PLACING CONCRETE: Concrete shall be handled from the mixer or transport vehicle to the place of final deposit in a continuous manner and as rapidly as practicable, until the given unit of operation, approved by the Contracting Officer, is completed. Concrete that has attained its initial set, or has contained its water content for more than one hour, shall not be deposited in the work. The concrete shall be deposited in the forms as nearly as practicable in its final position so as to avoid rehandling. Special care shall be exercised to prevent splashing the forms or reinforcement with concrete in advance of pouring. Special precautions shall be exercised in choice of aggregate and methods of placement so as to insure full bond of steel to concrete and to prevent voids at beam and column intersections. Concrete shall not be allowed to drop freely more than 5 feet in unexposed work, nor more than 3 feet in exposed work. Where greater drops are required, the method shall be approved by the Contracting Officer. The discharge shall be so controlled that the concrete may be effectively compacted into horizontal layers not exceeding 18 inches in thickness.

Where chutes are used for conveying concrete, they shall be of such size and design as will insure a continuous flow of concrete and prevent segregation of the aggregate. The chute shall be thoroughly cleaned before and after use. All waste material and the flushing water shall be discharged outside of the forms.

Concrete footings shall be placed upon undisturbed, clear, damp surfaces, free from mud, standing or running water. When the concrete is placed on dry soil, waterproof sheathing paper shall be laid over the earth surface to receive the concrete.

2-14. COMPACTION: Concrete, except when placed and compacted by pneumatic means, shall be placed in layers not over 18 inches deep and each layer shall be compacted with the aid of mechanical internal vibrating equipment supplemented by hand spading, rodding and tamping as directed by the Contracting Officer. Vibrators shall in no case be used to transport concrete inside the forms. The use of form vibrators will not

[redacted]

be permitted. Internal-vibrators shall maintain a speed of not less than 6,000 impulses per minute when submerged in the concrete. At least one spare vibrator shall be maintained as relief. The duration of vibration shall be limited to that necessary to produce satisfactory consolidation without causing objectionable segregation. The vibrator shall not be inserted into lower courses that have begun to set. Vibrators shall be applied at uniformly spaced points not farther apart than the visible effectiveness of the machine.

2-15. BONDING AND GROUTING: Before depositing new concrete on or against concrete which has set, the existing surfaces shall be thoroughly roughened and cleaned of all laitance, foreign matter and loose particles. Forms shall be retightened and the existing surfaces slushed with a grout coat of neat cement. The new concrete shall be placed before the grout has attained its initial set. Grout for horizontal construction joints shall be of cement and fine aggregate in the same proportion as the concrete to be placed, and shall be from 1/2 to 1 inch in thickness. Grout for setting column bases, wall plates, and other metal items, shall be composed of equal parts of sand and cement, with water sufficient to produce the required consistency.

2-16. CONCRETE FLOOR FINISHES: Concrete floor slabs shall be screeded and wood floated to the required level of the finished floors, as shown on the drawings.

2-17. FINISHES OF CONCRETE OTHER THAN FLOORS: Concrete for which no other finish is indicated or specified shall have all fins and rough edges removed.

2-18. PROTECTION AND CURING:

a. Protection Against Moisture Loss: Immediately after placing or finishing, concrete surfaces not covered by forms shall be protected from the loss of surface moisture for a period of not less than 14 days where a normal portland cement has been used, or 7 days where a high-early-strength portland cement has been used by covering with Kraft paper, quilts or burlap, lapped 4 inches at edges and ends, or by applying a curing compound. Kraft paper, if used, shall be sealed. Surfaces from which forms are removed before the curing period has elapsed, shall be protected as specified for surfaces not covered by forms. Quilts or burlaps shall be kept wet. Concrete surfaces covered by forms shall be protected from moisture loss by keeping forms thoroughly wet for the same period of time as specified above.

2-19. FORMS: Forms shall be constructed to conform to the shape, form, line, and grade required, and shall be maintained sufficiently rigid to prevent deformation under load.

a. Design: Joints shall be tight and leakproof and shall be arranged vertically or horizontally to conform to the pattern of the design. Where forms are placed in successive units for continuous

[redacted]

surfaces, they shall be fitted to accurate alignment so that the completed surface will be smooth and free from irregularities. If adequate foundation for shores cannot be secured, trussed supports shall be provided. Temporary openings shall be arranged in wall and column forms and where otherwise required, to facilitate cleaning and inspection. Lumber once used in forms shall have nails withdrawn, and the surfaces to be exposed to concrete carefully cleaned before re-use. All forms shall be so constructed that they can be removed readily without hammering or prying against the concrete. Inside forms shall be so designed that when stripped they may be readily removed through the openings provided. No forms shall be left in place except as shown on the drawings or with the approval of the Contracting Officer.

b. Form Ties: Bolts and rods used for internal ties which are to be removed, shall be coated with grease and so arranged that when the forms are removed, all metal except wire ties will be not less than 1 inch from any concrete surface. The design of form ties shall be subject to the approval of the Contracting Officer.

c. Joints: All corners and exposed joints in more than one plane, unless otherwise indicated on the drawings or directed by the Contracting Officer, shall be beveled, rounded, or chamfered by moldings placed in the forms.

d. Coating: Forms for exposed surfaces shall be coated with oil, applied before the reinforcement is placed. After oiling, any surplus oil on the form surfaces and any oil on the reinforcing steel, shall be removed. Forms for the unexposed surfaces may be thoroughly wetted with water in lieu of oiling immediately before the placing of concrete.

e. Removal: Forms shall not be disturbed until the concrete has adequately hardened. Shoring shall not be removed until the member supported has acquired sufficient strength to safely support its own weight and the load imposed on the shored member.

(1) Clamps: Tie-rod clamps that are to be entirely removed from the wall shall be loosened 24 hours after the concrete is placed and form-ties, except for a sufficient number to hold the forms in place, may be removed at that time.

(2) Timing: Under normal conditions, after placing concrete, the minimum waiting period before the forms may be stripped shall be governed by the following schedule; but the use of this schedule shall not operate to relieve the contractor of responsibility for the safety of the structure:

[redacted]

Stripping of Forms
(Minimum Waiting Period After Placing Concrete)

Structural Member	Average Temperatures Above 60° F	50° -60° F
Columns and walls	5 days	7 days
Side forms of girders & beams	5 days	7 days
Bottom forms of slabs 6 feet or less in clear span	5 days	9 days
Bottom forms of girders, beams, and slabs of span 6 feet and over	14 days	18 days

2-20. REINFORCING STEEL:

a. General: The reinforcement, fabricated to shapes and dimensions shown, shall be placed where indicated on drawings or reasonably required to carry out the intent of the drawings and specifications. Any changes shall be approved by the Contracting Officer and noted on the plans. Before placing, all reinforcement shall be thoroughly cleaned of rust, mill scale or coatings, which would reduce or destroy the bond. Reinforcement appreciably reduced in section shall not be used. Following any substantial delay in the work, previously placed reinforcement left for future bonding, shall be reinspected and cleaned. Reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on drawings shall not be placed. The heating of reinforcement for bending or straightening will be permitted only if the entire operation is approved by the Contracting Officer. In slabs and beams, reinforcement shall be spliced only as shown on drawings except as approved by the Contracting Officer. At all points where bars lap or splice including distribution steel, a wire-tied minimum lap of 40 bar diameters shall be provided, unless otherwise shown. Unless otherwise called for on the drawings, splices in columns shall be full butt-welded in accordance with requirements of the American Welding Society Code.

b. Design: Reinforcing details shown on the drawings shall govern the furnishing, fabrication, and placing of reinforcement, except as otherwise shown or specified, construction shall conform to the following requirements:

(1) Concrete Covering Over Steel Reinforcement: The con-

[redacted]

crete covering over steel reinforcement shall be as follows:

(a)	Beams	1-1/2"	outside of main steel
(b)	Columns	1-1/2"	outside of spiral steel*
(c)	Walls and slabs		diameter of reinforcement but not less than 3/4"
(d)	Footings	1-1/2"	outside of steel on sides
		2"	outside of steel on bottom

(2) Minimum Bar Size: Unless otherwise shown on the drawings, bars less than 3/8 inch diameter shall not be used in the work, except for stirrups, ties, and distribution steel.

(3) Spirals: Spirals shall be extended from tops of footings or floor slabs, to the level of the lowest horizontal reinforcement in the overhead slab, drop-panel, beam, or girder, unless otherwise shown. Spirals shall be secured by at least 3 spacer bars notched for the required pitch and anchored at both ends by 1-1/2 additional turns of spiral rod or wire. Splices in spiral rod or wire, where necessary, shall be made by welding. Vertical bars shall be wired to spiral at intervals not exceeding 18 inches.

(4) Shop Drawings: Shop detail drawings for all reinforcing steel shall be furnished for approval of the Contracting Officer.

c Supports: Reinforcement shall be accurately placed and securely tied at all intersections and splices with 18 gage black annealed wire, and shall be securely held in position by spacers, chairs or other approved supports during the placing of concrete, Wire tie ends shall point away from the form. The number of chairs or other supports used shall not be less than the following and shall be equally spaced.

(1) For Structural Slabs:

Clear Spans	Chairs per Bar
Spans up to 7 ft.	2
Over 7 ft. up to 17 ft.	3
Over 17 ft. up to 30 ft.	4

(2) For Slabs on Grade (Over Earth or Over Drainage Fill) and for Footing Reinforcement: The bars shall be supported on precast concrete blocks, spaced at the intervals required by the size of reinforcement used, to keep the reinforcement the minimum height specified above the underside of slab or footing.

* See Addendum

[redacted]

SECTION 3

BRICK; MASONRY

3-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances, and materials, and in performing all operations in connection with the installation of brick masonry, complete in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

3-02. APPLICABLE SPECIFICATIONS: The following Federal Specifications form a part of this specification:

QQ-B-71a.	Bars: Reinforcement (For) Concrete
SS-B-656.	Brick; Building (Common) Clay

3-03. MATERIALS:

a. Anchors: Design of anchors and web reinforcement shall be as shown on the drawings or otherwise approved by the Contracting Officer.

(1) Reinforcing steel rods as shown on the drawings shall conform to the requirements of Federal Specification QQ-B-71a, type B, deformed, grade 2, intermediate billet.

b. Brick:

(1) Common Brick: Common brick shall conform to the requirements of Federal Specification SS-B-656, class H or class M. Frog brick or brick having cores in excess of 1/2 inch in diameter or cores totaling in excess of 20 percent of the bed area shall not be used for exterior work.

(2) Reinforced Brick: Reinforcement shall be as shown on the drawings.

c. Mortar Materials: Mortar materials shall be as specified in the section, MORTARS: MASONRY, of these specifications.

3-04. HANDLING AND STORAGE: All masonry materials shall be handled in a manner to prevent undue breakage or chipping.

3-05. ERECTION:

a. General: All masonry shall be laid plumb, true to line, with level and accurately spaced courses, with corners plumb and true, and with each course breaking joint with the course below. Bond shall be kept plumb throughout. Units with greater than 12 per cent absorption shall be wetted before laying. Work required to be built in with the masonry, including anchors, reinforcement and accessories, shall be built in as the erection progresses.

b. Brick: Work not otherwise shown shall be common brick. All brick shall be damp when laid. Brick shall be shoved into place, not laid, in a full bed of unfurrowed mortar. All horizontal and vertical joints shall be completely filled with mortar when and as laid. Vertical joints shall be all of the same width except for inconspicuous variations required to maintain the bond. Back joints against concrete, metal or other units shall be slushed, grouted, or shoved full as the course is laid. All brickwork, not otherwise specified, shall be laid in class C mortar as defined in the section MORTARS: MASONRY of these specifications.

(1) Exposed or Face Work: Exposed exterior brick work shall conform to the sample approved by the Contracting Officer, and shall be laid in courses accurately spaced with a story rod, in common bond; that is, whole stretchers overlapping by half a brick, with a course of whole headers every sixth or seventh course and the bond extending the full thickness of the wall for 8" walls; and as shown on the drawings for thicker walls. Exterior walls shall be laid in class C mortar as defined in the section MORTARS; MASONRY of these specifications.

(2) Joints: Exposed brickwork shall be laid with joints flush, or as directed by the Contracting Officer. Unless otherwise indicated on the drawings or specified, mortar joints shall be approximately 3/4 inch thick for brick joints with reinforcing rods and 1/2 inch thick for all other brick joints. All joints shall be formed with suitable tools. Joints shall be tooled in such manner as to squeeze the mortar back into the joints completely filling any cracks in the joint. No tooling shall be done until after the mortar has taken its initial set.

(3) Reinforcement: Reinforcing rods shall be placed as called for on the drawings.

c. Unfinished Work: Any brick masonry which is started should be completed in one operation.

d. Curing and Protection: Surfaces of masonry not being worked on shall be properly cured and protected at all times during the construction operation. At such time as rain is imminent and the work is discontinued, the tops of exposed masonry walls and similar surfaces shall be covered with a strong waterproof membrane well secured in place. Brick panels shall be completed at least 30 days before final test.

[redacted]

SECTION 4

MORTARS; MASONRY

4-01. SCOPE: This section of the specifications covers mortars to be provided in connection with all types of masonry work as specified under other sections of these specifications and as referenced therein by their class designations.

4-02. APPLICABLE SPECIFICATIONS: The following specifications form a part of this specification:

a. Federal Specifications:

SS-C-192. Cement; Portland
SS-L-351. Lime; Hydrated (for) Structural Purposes

b. American Society for Testing Materials Serial Designations:

C 144-44. Aggregate for Masonry Mortar
C 161-44T. Mortar for Reinforced Brick Masonry

4-03. GENERAL: All cementitious material shall be delivered to the site in manufacturers' standard packages. Mortars in which portland and other quick setting cements are used shall be prepared in batches of the volume that will be used before initial set takes place, and in no case longer than 45 minutes before delivery to the masons' mortar boards at points of use. Retempering will not be permitted.

4-04. MATERIALS: Cement that has been stored in sacks for more than 6 months shall not be used without retesting.

a. Portland Cement: Portland cement shall conform to the requirements of Federal Specification SS-C-192. Unless otherwise specified, portland cement shall be type I.

b. Lime: Lime shall be hydrated lime conforming to the requirements of Federal Specification SS-L-351, Type F, with the further requirement that the total free (unhydrated) calcium oxide (CaO) and magnesium oxide (MgO) in the hydrated products shall not exceed 8 percent by weight, calculated on the "as received" basis.

c. Sand (Fine Aggregate): Sand shall be of an acceptable color and shall conform to the requirements of American Society for Testing Materials Standard Specification C 144-44. Sand shall be graded from fine to coarse within the following limits:

[redacted]

(1) Sand for Mortar Joints Over 1/4 inch in Thickness:

Sieve No.	Percent of Sand Retained (By Weight) Maximum	Minimum
4	0	
8	10	0
16	40	15
30	65	35
50	85	75
100	98	95

d. Water: Water shall be clean and free from injurious amounts of oil, acids, soluble salts and organic impurities.

4-05. SAMPLES: Samples of all materials specified shall be submitted for approval when required by the Contracting Officer.

4-06. TESTING: The customary production plant testing of cementitious materials will be the responsibility of the contractor. Certified copies of the plant test results shall be furnished to the contracting Officer when required. All other testing required will be done by the Contracting Officer without expense to the contractor. The testing of sand for gradation and other properties shall conform to the requirements of A.S.T.M. Serial Designation C 144-44. The testing of mortar shall conform to the requirements of A.S.T.M. Serial Designation C 161-44T.

4-07. STORAGE AND PROTECTION: Cement, lime, and any admixture materials, immediately upon delivery to the site, shall be stored in weatherproof sheds, or upon platforms raised free of the ground and effectively protected by tarpaulin covers until used. No cementitious or other material that has become caked and hardened from absorption of moisture will be permitted in the work.

4-08. CLASSES AND PROPORTIONING OF MORTARS: The proportioning of the various classes of mortars shall be as specified below. The proportions of cement specified are the minimum. Where the sand, which is locally or readily obtainable, does not produce a mortar having the crushing strength herein specified for the particular class of mortar required but is in all other respects satisfactory, the sand content shall be decreased to the extent required to obtain that strength with related density, bonding value, and other properties.

Classification and Proportioning of Mortars (See Note 1)

Designation of Mortar	Portland Cement (Sack)	Slag Cement (Sack)	Masonry Cement (Sack)	Lime Hydrated (Sack)	Lime Putty (Cu.Ft.)	Sand, Dry Volume (Cu.Ft.)	Sand, Dry Weight (Lbs.)
Class C	1	—	—	—	1-1/4	6	480

[redacted]

NOTE 1: The volume units of mortar materials shall be taken as follows:

Portland Cement	- one sack, 94 pounds net........	1 cubic foot
Hydrated Lime (dry)	- one sack, 50 pounds net........ (contains 1-1/4 cu. ft.)	1 cubic foot
Sand - 80 pounds dry, or 85 pounds damp		1 cubic foot

4-09. STRENGTH OF MORTARS: Mortar shall possess not less than 2500 pounds psi minimum strength values, when tested in cubes or cylinders at the end of a 28 day aging period.

4-10. PREPARATION AND STORAGE OF LIME PUTTY: Lime putty, when required, shall be made from hydrated lime. The dry hydrated lime shall be mixed with water to form a stiff, plastic putty. When putty must be stored for more than 24 hours before use in the mixing of mortar, suitable precautions shall be taken to protect it from exposure to sun and prevent excessive evaporation.

4-11. MIXING OF MORTARS: Mortars shall be machine-mixed in an approved type of mixer in which the quantity of water can be accurately and uniformly controlled; however, for work requiring only small batches of mortar or grout, or when specifically approved by the Contracting Officer, mortar may be mixed by hand. The mixing time shall be not less than 5 minutes, approximately 2 minutes of which shall be for mixing the dry materials and not less than 3 minutes for continuing the mixing after the water has been added. The proportioning of materials for the mortar required, shall be as given in the table showing classification and proportions of mortar. Where hydrated lime is used for mortar requiring a lime content, the contractor will have the option of using the dry-mix method of first converting the hydrated lime into a putty. Hand-mixing shall be done in a tight mortar-mixing box. Where the dry-mix method is employed, the materials for each batch shall be well raked and turned over together before the water is added, until the even color of the mixed materials indicates that the cementitious material has been thoroughly distributed throughout the mass, after which the water shall be gradually added until a thoroughly mixed mortar of the required plasticity is obtained.

a. Mortar Grout: Mortar for grouting and poured fills shall have the quantity of water increased to produce the consistency required for pouring and shall be continuously stirred to prevent the segregation of the aggregate.

Grout for sealing joints between precast segments shall be so placed as to completely fill the volume fo the joint, securely bond the steel reinforcement, and prevent the formation of voids. The contractor shall submit the procedure for grouting to the Contracting Officer for approval.

[redacted]

SECTION 5

STRUCTURAL STEEL

5-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances, and materials, and in performing all operations in connection with the installation of structural steel complete, in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

5-02. APPLICABLE SPECIFICATIONS AND CODES. The following specifications and codes form a part of this specification:

a. Federal Specifications:

QQ-S-741. Steel, Structural (Including Welding) and Rivet; (for) Bridges and Buildings.

WW-P-406. Pipe; Steel and Ferrous-Alloy (for) Ordinary Uses (Iron-Pipe Size).

b. American Institute of Steel Construction Publications:

Code of Standard Practice for Steel Buildings and Bridges.

Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

c. American Welding Society Code:

Arc and Gas Welding in Building Construction.

5-03. GENERAL: The current rules and practices set forth in the Code of Standard Practice for Steel Building and Bridges, and the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings of the American Institute of Steel Construction, shall govern this work, except as otherwise noted on the drawings or as otherwise specified. Welding shall be in accordance with the current Code for Arc and Gas Welding in Building Construction of the American Welding Society.

a. Shop Drawings: Shop drawings, in triplicate, shall be submitted to the Contracting Officer for approval. Material fabricated or delivered to the site before the approved shop drawings have been received by the contractor shall be subject to rejection by the Contracting Officer.

b. Mill Reports: The contractor shall furnish, without extra cost to the Government, two certified copies of all mill reports covering the chemical and physical properties of the steel used in the work under this specification.

[redacted]

c. Responsibility for Errors: The contractor alone shall be responsible for all errors of fabrication and for the correct fitting of the structural members shown on the shop drawings.

5-04. MATERIALS:

a. Structural Steel: Structural steel shall conform to the requirements of Federal Specification QQ-S-741, Type I or II, Grade B, Class 1.

b. Anchor Bolts: All anchor bolts shall conform to the requirements for structural steel.

c. Sleeves: Pipe sleeves for anchor bolts shall conform to the requirements of Federal Specification WW-P-406.

5-05. INSPECTION AND TESTS: The material to be furnished under this specification shall be subject to inspection and tests in the mill, shop, and field by authorized Government inspectors. Inspection and tests will be conducted without expense to the contractor; however, inspection in the mill or shop shall not relieve the contractor of his responsibility to furnish satisfactory materials, and the Government reserves the right to reject any material at any time before final acceptance of the building when, in the opinion of the Contracting Officer, the materials and workmanship do not conform to the specification requirements.

Two test specimens for determining the average yield point stress shall be obtained from each column between each floor of buildings 2 and 6. Two test specimens shall also be obtained from one front and one rear wall girt for each floor of buildings number 2 and 6. Each test specimen shall be at least eighteen inches long. Each specimen shall be numbered and a drawing or sketch shall be made to locate the position in the building of each member from which the specimen was obtained. The specimens and location sketch shall be forwarded by the contractor to a testing laboratory as directed by the Contracting Officer.

5-06. DESIGN: The members and connections for all portions of the structures shall be as indicated on the drawings. In the event that it is deemed necessary to modify or change any members or connections, drawings shall be submitted to the Contracting Officer for approval before any material is fabricated.

5-07. WORKMANSHIP:

a. Riveted Connections: Unless otherwise shown on the drawings, all connections shall be riveted with 1" ø rivets in 1-1/16" diameter holes. All connections shall be in accordance with the requirements of the A.I.S.C. Specifications.

[redacted]

b. Welded Connections: All welded connections called for on the drawings shall be shop welds. Groove welds shown on the drawings shall be full penetration groove welds.

The welding electrodes shall be A.W.S. No. 110, or electrodes having equal physical properties in tension, yield point and ductility.

All welds shall be stress relieved.

c. Holes: Holes shall not be made or enlarged by burning, nor will the burning of unfair holes in the shop or field be acceptable. Holes in base or bearing plates shall be drilled. Grout holes shall be provided in column bearing plates where shown on drawings.

d. Nuts: Nuts shall be of an approved self-locking type; or the bolt threads shall be upset to prevent backing off of the nuts.

5-08. ERECTION:

a. Drift Pins: Drift pins may be used only to bring together the several parts; they shall not be used in such manner as to distort or damage the metal.

b. Gas Cutting: The use of a gas cutting torch in the field for correcting fabrication errors will not be permitted on any major member in the structural framing. Its use will be permitted on minor members when the member is not under stress, and then only after the approval of the Contracting Officer has been obtained.

c. Base Plates and Bearing Plates: Column base plates and large bearing plates shall be supported on steel wedges or shims until the supported members have been plumbed, following which the entire bearing area shall be grouted solid with equal parts of portland cement and sand.

5-09. PAINTING AND CLEANING: Structural steel shall be painted as specified in Section 8 PAINTING; PROTECTIVE, of these specifications. Unpainted parts which will be in contact with concrete shall be thoroughly cleaned of rust, mill scale, or coatings which would reduce or destroy the bond.

[redacted]

SECTION 6

CORRUGATED ASBESTOS-CEMENT AND V-BEAM METAL SIDING

6-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances, and materials, and in performing all operations in connection with the installation of wall siding complete, in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

6-02. GENERAL: Siding shall be either corrugated asbestos-cement type or V-beam metal type as indicated on the drawings.

6-03. MATERIALS:

a. Corrugated, Asbestos-Cement Siding: Corrugated asbestos-cement siding shall conform to the requirements of Federal Specification SS-R-524, Type I.

b. V-beam Metal Siding: Metal siding shall be protected type metal siding, as manufactured by the H. H. Robertson Company, Pittsburgh, Pennsylvania, or a similar product of other manufacturers equal in strength, stability, and section modulus to the product above specified. The metal siding shall be made using black steel sheets meeting requirements of A.S.T.M. tentative specification A246-44T as a base. The sheets shall be No. 18 gage 29" wide, with 1-3/4" deep corrugations, 5.3" wide. Minimum side-lap shall be 6", and end-laps as shown on the drawings and only where shown. Coating shall be clean, dry and non-sticking, and the steel shall be completely protected against corrosion including edges and holes cut in the field. Sheets shall be of length indicated on the drawings.

c. Accessory Materials:

(1) Fasteners and Accessories: Fasteners and accessories including bolts and screws, shall conform to details as shown on the drawings.

(2) Handling and Storage: Material shall be carefully handled to avoid damage to coatings. Sheets shall be stacked on firm level supports to avoid distortion. Sheets shall be sorted and piled accordingly to size. All material shall be protected from traffic and from contact with dirt, and mud stain of any kind.

(3) Manufacturers Detailed Drawings: All drawings shall be submitted in triplicate to the Contracting Officer for approval. These drawings shall show the complete detailed method of application. The shape and type of fastener for each type of structural support shall be indicated. The length and width of all sheets shall be clearly indicated. Field cutting shall be minimized.

[redacted]

(4) Sheets: Sizes of sheets shall be as required for the conditions indicated on the drawings.

6-04. INSTALLATION:

a. Fasteners: Fasteners for securing V-beam sheets to structural steel supports shall be spaced as shown, with at least two to a sheet at each support. Standard Sidelap bolts shall be placed in each sidelap 12" o.c. V-beam metal and corrugated asbestos-cement sheets shall be anchored to the supports by means of anchor bolts set into the concrete wall as shown on the drawings.

b. Sheets: V-beam protected metal sheets shall be laid as indicated on the drawings. Strap fasteners where used shall be tight, metal strips placed snug against structural steel supports. End laps shall occur over structural members only where shown on the drawings, and fasteners shall pass through all overlapping sheets. Holes for fasteners shall be punched from the outside with sharp pointed punches. Nuts on bolts shall be tightened sufficiently to bring the bolt head tightly against the sheets.

[redacted]

SECTION 7

CARPENTRY

7-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances, and materials, and in performing all operations in connection with the installation of carpentry, complete, in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

7-02. APPLICABLE SPECIFICATIONS: The following specifications form a part of this specification:

a. Federal Specifications:

FF-B-561.	Bolts, Lag; Steel (Lag-Screws)
FF-B-571a.	Bolts; Nuts; Studs; and Tap Rivets (and Materials for Same)
FF-N-101.	Nails; Spikes; Staples: and Tracks.
FF-S-111.	Screws; Wood
MM-L-751c.	Lumber and Timber; Softwood

b. Department of Commerce Publications:

R16-39	Lumber-American Lumber Standards for Softwood Lumber

7-03. MATERIAL; LUMBER AND WOODWORK: Lumber shall conform to the requirements of Federal Specification MM-L751c. Lumber for the various uses shall be pine or fir and of the grade indicated on the drawings.

7-04. MATERIALS OTHER THAN LUMBER:

a. Bolts and Nuts: Bolts and nuts shall conform to the requirements of Federal Specification FF-B-571a, Class B.

b. Lag Screws: Lag screws shall conform to the requirements of Federal Specification FF-B-561.

c. Nails: Nails shall conform to the requirements of Federal Specification FF-N-101, or may be the drive screw or spiral type of standard manufacture.

7-05. LUMBER:

a. Grading and Selection: Grading of lumber of the various species shall conform to the requirements of Federal Specifications MM-L-751. Selection of lumber for the use intended shall be approved by the Contracting Officer.

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b. Sizes and Patterns: Lumber shall be surfaced four sides and the dressed sizes of yard and structural lumber shall conform to the requirements of Department of Commerce Simplified Practice Recommendation R16-39.

c. Moisture Content: Unless otherwise permitted by the Contracting Officer, lumber shall be either air-dried or kiln-dried, and the moisture content shall not exceed 19 percent.

d. Storage: Lumber delivered to the site shall be carefully piled off the ground in such manner as to insure proper drainage, ventilation, and protection from the weather.

7-06. ANCHORS: Anchors shall be installed where specified, shown, or required to anchor carpentry to masonry or concrete.

7-07. TEMPORARY COVERS: Temporary timber covers shall be provided over openings in the structured as called for on the drawings.

7-08. ACCESS FACILITIES: The contractor shall provide ladders, stairways or ramps before and after the test such that access is provided for the installation, inspection reading, and removal of the instruments used in the various buildings. The contractor shall also provide either a continuous or a movable scaffold before and after the test such that access is provided to each survey point as shown on the drawings. The contractor shall develop said stairways, ramps, and scaffolds with knowledge of the use of the facility and the entire scheme shall be subject to the approval of the Contracting Officer.

SECTION 8

PAINTING; PROTECTIVE

8-01. SCOPE: The work covered by this section of the specifications consists in furnishing all plant, labor, equipment, appliances, and materials, and in performing all operations in connection with protective painting complete, in strict accordance with this section of the specifications and the applicable drawings, and subject to the terms and conditions of the contract.

8-02. APPLICABLE SPECIFICATIONS: The following Federal Specifications form a part of this specification:

TT-P-20. Paint, Blue Lead Base; Basic Sulphate, Linseed Oil, Ready Mixed.

TT-P-86. Paint, Red Lead Base; Linseed Oil Ready Mixed.

8-03. GENERAL: Unless otherwise specified, all exterior and interior ferrous metal except protective siding, reinforcing steel, bolts, rough hardware, and such parts of the structural steel members as will be in contact with concrete, shall be given a shop coat of protective paint. Paint shall conform to the requirements below, subject to the approval of the Contracting Officer.

8-04. MATERIAL:

a. Exterior Type Paint: Paint for metal except as otherwise noted shall conform to the requirements of Federal Specification TT-P-20 or TT-P-86.

b. Special Paint: Certain areas may be designated to be painted for photographic purposes. The areas to be painted and the type of paint to be used shall be as directed by the Contracting Officer.

8-05. APPLICATION: Surfaces to be painted shall be thoroughly cleaned of scale, dirt, and rust by the use of steel scrapers, wire brushes, sand blast, or other equally suitable tools or methods. Oil and grease shall be removed with benzine or other suitable solvent. Paint shall be kept well stirred while it is being applied. No paint shall be used after it has caked or hardened. Paint shall be well worked into all joints and corners. Paint shall not be applied to damp surfaces.

[redacted]

SECTION 9

MISCELLANEOUS

9-01. INSTRUMENTATION: The work covered by this section consists of the furnishing and installation of all sleeves and anchors necessary to support the appliances used in instrumentation. Specific detail requirements shall be furnished by the Contracting Officer. The contractor shall furnish and install all survey anchor bolts as shown on the contract drawings, and he shall provide access before and after test as previously specified in Section 7-08.

9-02. LIVE LOAD: The work covered by this section consists in furnishing all labor and materials required for the installation of the simulated live load as detailed in the contract drawings. Certain areas, as designated by the Contracting Officer, shall be left open for instrumentation purposes. The live load shall be distributed over the remaining area of the floor so as to average 100 p.s.f. over the entire area. After the test the contractor shall furnish labor and equipment for the clearing of such floor areas as may be designated by the Contracting Officer.

9-03. SHELTER DOORS: The contractor shall furnish, fabricate, deliver and install the following:

(a) Structural steel doors as shown on the drawings and as specified. He shall place the doors in position and make all necessary adjustments in order to put them in satisfactory operating condition.

(b) One hollow metal door with hardware and other details as called for on the drawings, similar and equal to Dahlstrom Metallic Door Company. The contractor shall so set the door in place as to insure a tight fit against the sponge rubber gasket.

9-04. CORRUGATED STEEL SHEETS FOR UNDERGROUND TEST SHELTER: The contractor shall furnish and erect the steel sheets as called for on the drawings.

Steel Sheets shall be ARMCO Improved Multi-plate or equal, conforming to Federal Specification QQ-S-741. The plates shall be punched with 13/16 ø holes for 3/4 ø bolts as shown on drawings. The plates shall be assembled at the site according to the manufacturer's recommendations. Uniform support shall be obtained by thoroughly tamping the back-fill material to a depth of 3/4 of the pipe diameter.

9-05. MEMBRANE WATERPROOFING: The contractor shall furnish and install the 2-ply membrane waterproofing in connection with the underground test shelter where called for on the drawings, and where ordered by the Contracting Officer. The waterproofing fabric and asphalt for mopping coats shall conform to the requirements of Federal Specification HH-C-581 and SS-A-666, Type III respectively.

9-06. SEALING STRIPS: Shall be furnished and placed between buildings and at other locations shown on the drawings.

The fabric shall be fibreglass woven into Hess-Goldsmith White Fabric No. 182 or equal, thickness .013 inches, 12.4 oz. per square yard; min. ave. breaking strength 440 lbs. per inch on warp and 400 lbs. per inch on fill.

Sealing strips between buildings shall be made of two layers of fabric at the front wall. All other joints shall be sealed with a single layer of fabric.

Edges of fabric shall be rolled over hemp twine conforming to Federal Specification T-T-901a, and clamped to the structure by structural steel angles, drilled for anchor bolts where attached to concrete, or for bolting to structural steel. A one-eighth inch plywood cushion strip shall be provided for clamping directly to the concrete.

Filler blocks under fastening clamps shall be provided where connections are made to V-beam metal or currugated asbestos siding.

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SECTION A1

ADDENDUM NO. 1

TO

TECHNICAL SPECIFICATIONS

1. PAGE A6-2-12 SECTION A6-2-20.b (1) Concrete Covering Over Steel Reinforcing:
   Change line (b) to read "Columns - as shown on drawings"

2. PAGE A6-5-2 SECTION 5-05. INSPECTION AND TESTS:
   Omit the second paragraph and insert:
   "Two test specimens for determining the average yield point stress shall be obtained from each rolling of each structural steel section to be used for columns in buildings number 2 and 6. Two test specimens shall also be obtained from each rolling of each structural steel section to be used for wall girts in buildings number 2 and 6. Each test specimen shall be at least eighteen inches long. Each specimen shall be numbered and a drawing or sketch be made to locate the position in the building of all members for which the specimen is representative. The specimens and location sketch shall be forwarded by the contractor to a testing laboratory as directed by the Contracting officer."

3. PAGE A6-7-1 SECTION 7-02. APPLICABLE SPECIFICATIONS:
   To paragraph b. Department of Commerce Publications add:
   CS 45-42 Douglas Fir Plywood, Commercial Standard

4. PAGE A6-9-1 SECTION 9-02. LIVE LOAD:
   Add the following:
   Such special precautions shall be taken in the vicinity of the open areas to prevent shifting of the live load during the test and to protect the instruments and instrument supports from damage as may be required by the Contracting Officer.

5. PAGE A6-9-1 SECTION 9-03. SHELTER DOORS:
   Add the following:
   (c) Two wood doors with hardware and other details as called for on the drawings. These doors shall conform to Department of Commerce Publication CS120-46, Standard Stock Ponderosa Pine Doors and shall be made in accordance with the latest standard specification of the National Door Manufacturers' Association. The doors shall be of the design and size indicated on the drawings and they shall be fabricated in accordance with the best practice of the trade, with all joints properly formed, tightly fitted and glued. The doors shall receive such treatment to prevent deterioration as may be required by the Contracting Officer. Top and bottom

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edges shall receive one coat of spar varnish at the factory before shipment.

Doors shall be hung by the contractor as indicated on the drawings and to provide 1/16 inch clearance on the sides and top and 3/16 inch clearance over the thresholds. The contractor shall also set the door to ensure a tight seal on all four edges as shown on drawings.

Doors shall be hung and trimmed with hardware conforming to Federal Specification FF-H-106 and FF-H-116b and as shown on the drawings.

6. PAGE A6-9-1 SECTION 9-04 CORRUGATED STEEL SHEETS FOR UNDERGROUND TEST SHELTER

To the second paragraph add:

"Before backfilling, the vertical diameter of the pipe shall be elongated by strutting in accordance with the manufacturers recommendations so as to obtain three percent elongation of the vertical diameter. Struts shall not be removed until the entire operation of backfilling is complete."

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APPENDIX 7

DETAILED CONSTRUCTION DRAWINGS

The drawings listed here are reproduced in WT-60 (REF), Parts I and II, and may be seen by referring to it. The page numbers given here are for that report.

Drawing No.	Sheet No.	Title	Page
60-09-06	1	Key Plans and Elevations of Multistory Buildings	61
"	2	Buildings Nos. 1 and 7, Concrete Plans and Details	62
"	3	Buildings Nos. 1 and 7, Interior Wall Elevations, Sections and Details . . .	63
"	4	Building No. 1, Test Panels--Elevations and Details	64
"	5	Building No. 7, Test Panels--Elevations and Details	65
"	6	Buildings Nos. 2 and 6, Concrete Plans and Details	66
"	7	Buildings Nos. 2 and 6, Structural Sections and Details	67
"	8	Buildings Nos. 2 and 6, Wall Elevations, Sections and Details	68
"	9	Buildings Nos. 3 and 5, Concrete Plans and Details	69
"	10	Building No. 4, Concrete Plans and Details .	70
"	11	Buildings Nos. 3, 4, and 5, Wall Elevations, Sections and Details	71
"	12	Buildings Nos. 3, 4, and 5, Column Schedules, Wall Elevations and Details . . .	72
"	13	Location Plan for Survey Points and Access Ways	73
60-09-11	1	Underground Test Shelter--General Plan, Sections and Details	74
"	2	Underground Test Shelter--Concrete and Reinforcing Details	75
"	3	Underground Test Shelter, Door Details .	76

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